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Engineering News-Record

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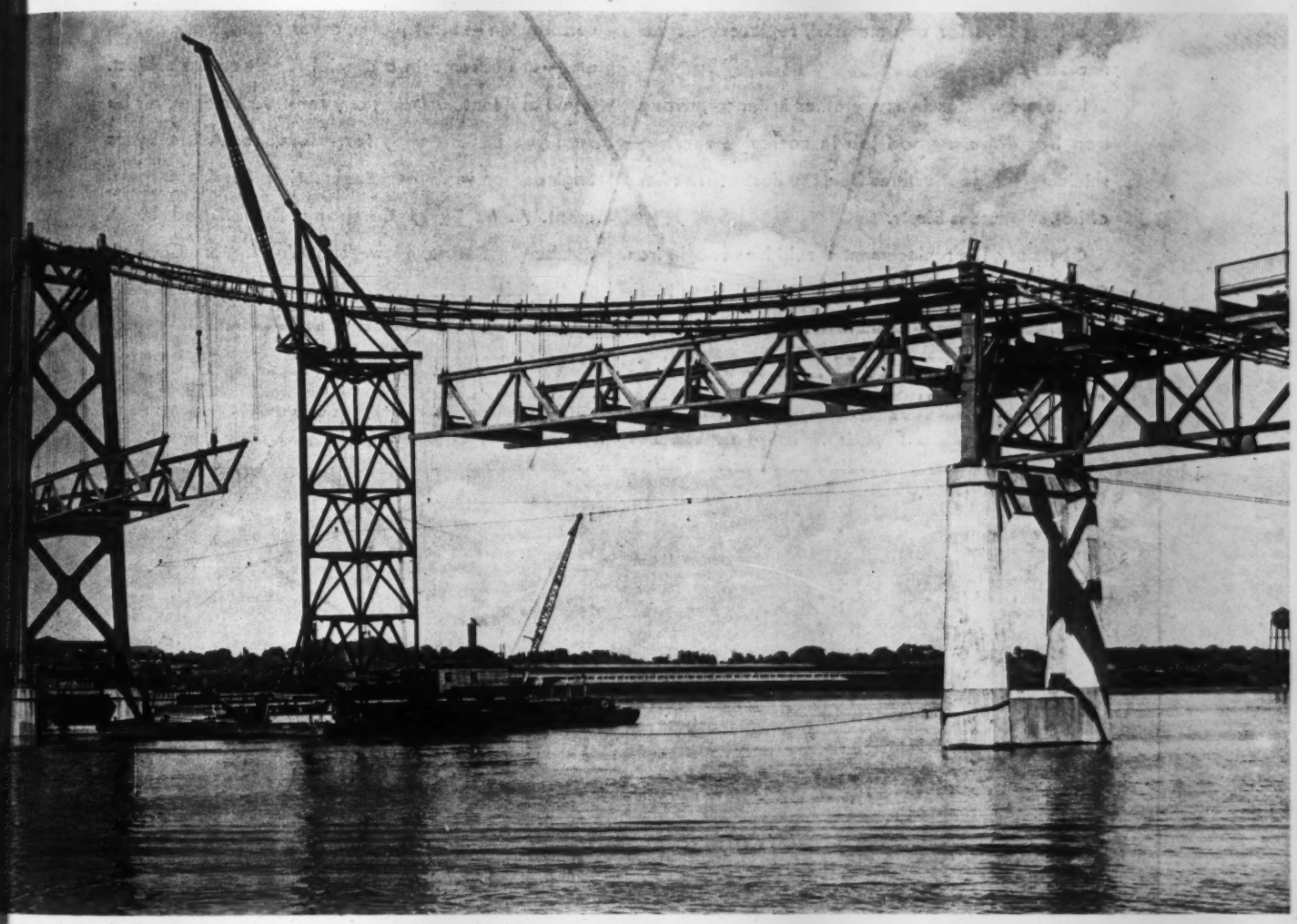

Stiffening truss erection by floating derrick on the Davenport Bridge over the Mississippi River

IN THIS ISSUE: Building the Davenport Bridge Across the Mississippi River . . . Cast-in-Place Short Piles Show High Test Results . . . Coefficients for Beam Deflections Under Various Loading Arrangements . . . Express Highway Without Grade-Crossings at St. Louis . . . Montreal to Build New Intake for St. Lawrence River Water . . . Primary Sewage-Treatment Plant Includes Magnetite Filters . . . Earthquake-Resistant Construction Applied to California Schools . . . Slag-Cement Blend Concrete Tried at Norris Dam.

61 YEARS in a Railroad Tunnel

● When the Oswego Tunnel on the New York Central was built in 1872, according to the records, wrought iron was used for cover plates. 61 years later, which is a long time for any material, replacements were necessary.

Here, as well as in many other instances, wrought iron has done a good job in corrosive service—shielding bridge structures from the destructive effects of hot, sulphurous blasts.

Obviously, the replacement should be wrought iron. Consequently, 13.5 tons of Byers Wrought Iron Plates were used and thus the second generation of wrought iron started in where the first generation left off.

The most important uses for wrought iron in bridge construction — blast plates, cover plates, deck plates, etc. — are covered in a special report on the subject. If you are contemplating any construction or repair be sure you have a copy for reference. Ask a Byers Engineer or write our Engineering Service Department. A. M. Byers Company, Established 1864. Pittsburgh, Boston, New York, Washington, Chicago, St. Louis, Houston.

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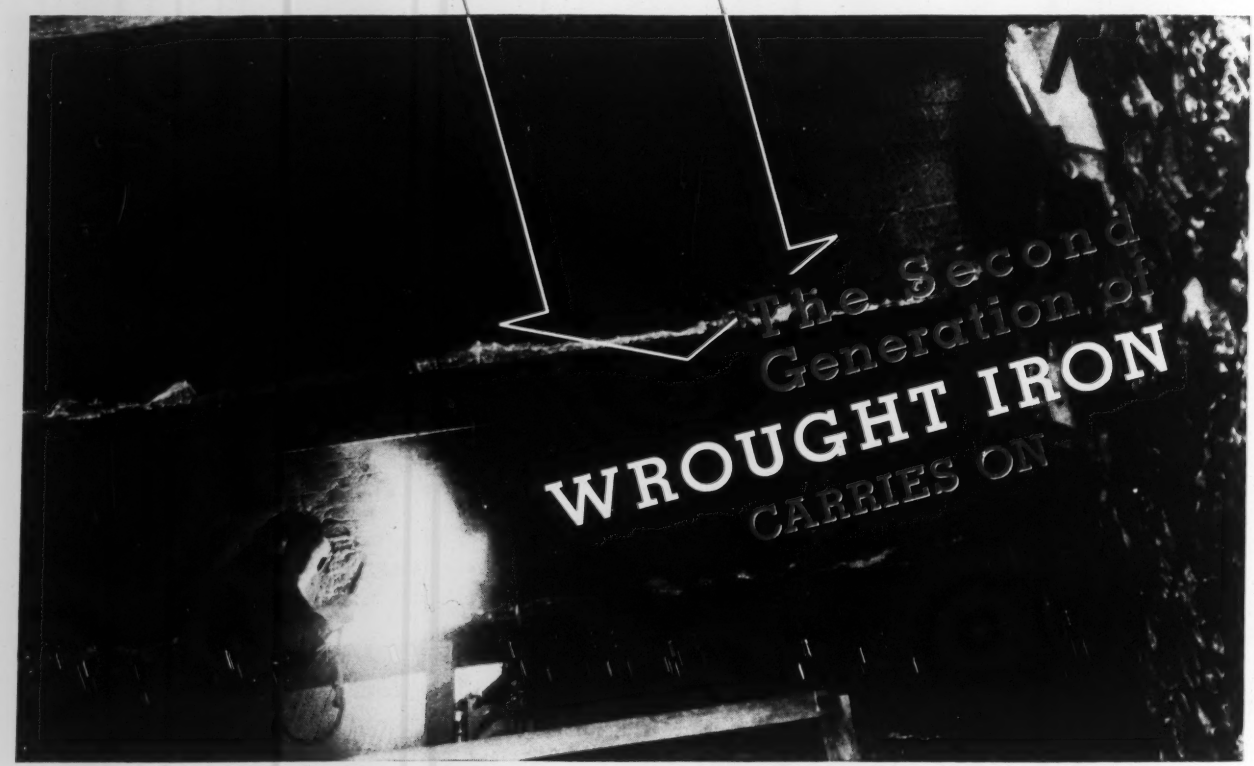

A MOMENT WITH THE PUBLISHERS

WITH the advent of this issue readers of Engineering News-Record will find themselves on the eve of another Christmas Day And so may I lay aside, for the moment, every concern that ordinarily might command the consideration of this page and devote it to the happier and more seasonable purpose of wishing to the men of the construction industry—engineers, architects, contractors, subcontractors, manufacturers and distributors of materials and equipment, and all the rest,

* *A VERY MERRY CHRISTMAS* *

Willard Chevalier

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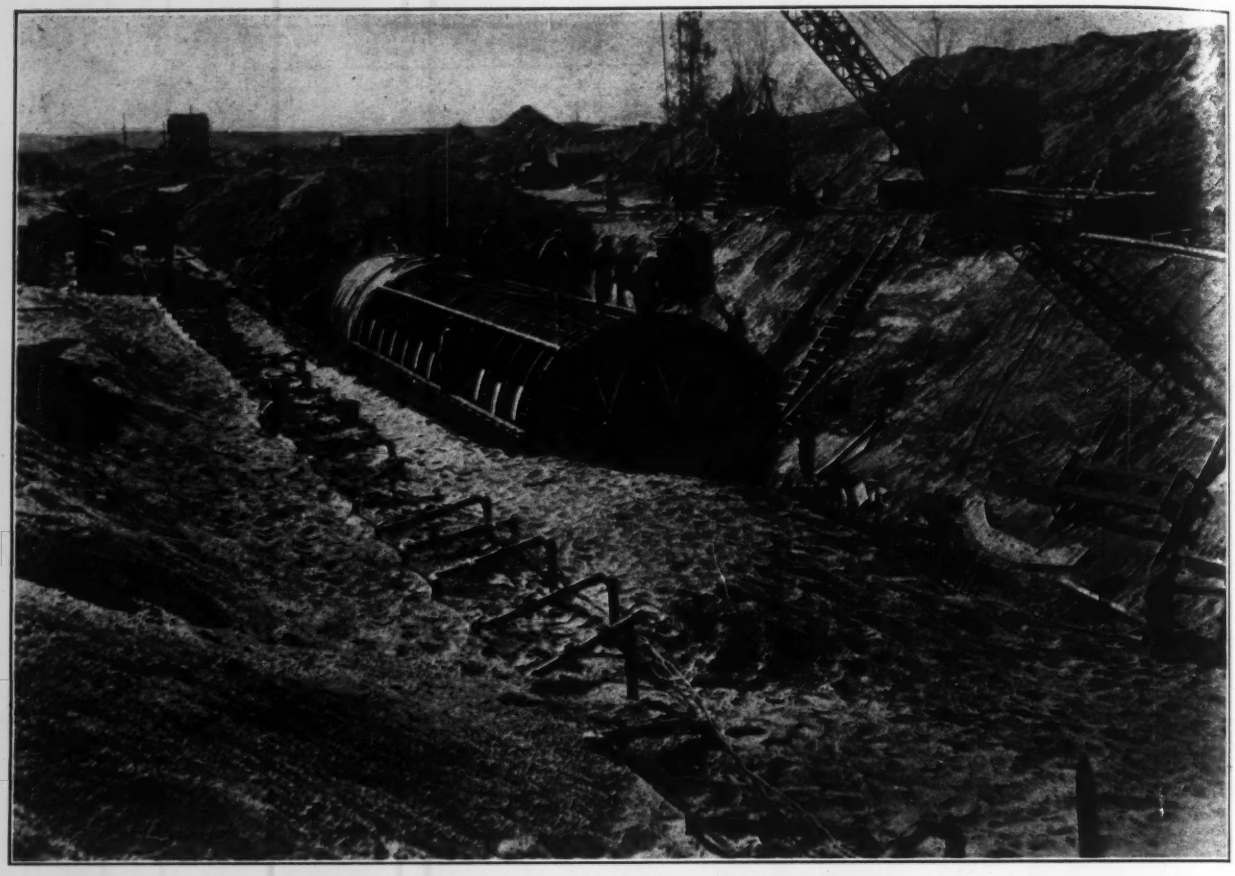

Siphon under the South Platte River at Paxton, Nebraska

Best Wishes for

A Very Merry Christmas

and

A Happy and Prosperous New Year

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In the News:

THE NEW $13,000,000 combined highway and railway bridge over the Mississippi River just above New Orleans was opened to traffic last week. With the long railway approaches made necessary by 135-ft. underclearance over the navigation channel, the bridge has a total length of nearly 4½ miles.

SECRETARY ROPER has called a safety conference to be held in Washington this week. He proposes to consider all kinds of accidents, pointing out that while highway accidents are most in the public eye, they are responsible for only one-third of the fatalities.

THE MEETING of the American Shore and Beach Preservation Association in Washington Monday was made the occasion for a concerted effort to arouse national interest in the defense of the New Jersey coast against erosion.

CURRENT REPORTS that a portrait of John F. Stevens is to be used on a U. S. postage stamp are in error. Long-established custom in this country requires that a man must be dead before his picture is used on a postage stamp, and Mr. Stevens is very much alive! The stamp on which his portrait is to be used is a Canal Zone stamp, one of a series that the Canal Zone has been issuing for some time past honoring engineers and others who served there at one time or another. The date of issue is not known.

CONTINUED INCREASE in the country's steel sheet and strip capacity is evident in the plans of Jones & Laughlin to invest $25,000,000 in such facilities in the Pittsburgh area. The company, fourth largest steel manufacturer in the country, has not been engaged in sheet manufacture heretofore.

CONSTRUCTION or demolition contracts have been let on 47 of PWA's 50 low-rent housing projects. Nine projects are under construction, and three are under general contract; foundations are under way on 23, and demolition on 12 others.

THE REPORT OF LOUIS R. GLAVIS lists the savings effected on PWA work through the efforts of his division of investigations. Some $35,000,000 is listed as saved by the detection of irregularities in contract bids and awards (dirty work at the crossroads, as one Washington newsman described it), while another $30,000,000 was saved by eliminating approved projects which further investigation proved to be economically unsound.

EXCAVATION at Grand Coulee Dam has reached 17,500,000 cu.yd., nearly three times the amount at Boulder Dam. Concreting, which has just begun, is planned to attain a volume of 12,000 cu.yd. average per day by next spring. At Boulder Dam 6,000 yd. daily was the average, with a record pour of 10,642 yd.

In This Issue:

PUTTING THEORY INTO PRACTICE, the cities of California have gone into the construction and reconstruction of their schools on a grand scale, to make them resistant to earthquakes. The general program being put into effect, in accordance with state law requirements, is outlined by C. H. Kromer, principal structural engineer of the state division of architecture.

THE NEW BRIDGE over the Mississippi River, serving the Tri-Cities group of Moline and Rock Island, Ill., and Davenport, Iowa, and built by the Davenport Bridge Commission, includes a 740-ft. span suspension bridge and a series of continuous-truss spans. Its erection was characterized by the use of a floating tower derrick, which placed the towers and stiffening trusses of the suspension-bridge portion.

COMPARATIVE TESTS of short poured-in-place concrete piles and spread footings are reported, showing small relative settlements for the piles and in some cases the possibility of a saving in materials costs.

TABLES OF COEFFICIENTS that will permit vertical deflections of beams to be determined with sufficient accuracy for all practical purposes have been developed by Robins Fleming and are published here.

ST. LOUIS is completing the first grade-separated express highway in the Middle West, a three-mile five-lane road that will give residents of western suburbs easy access to the central business district. Two-thirds of the length is in Forest Park, and the remainder is in a new right-of-way traversing a blighted area of old residences. Several interesting bridge structures are included.

FACED with a water-supply problem involving both low quality and insufficient quantity, Montreal has taken bids for a new water intake in the St. Lawrence River, including six lines of 7-ft.-diameter concrete pipe over 2,000 ft. long. The water-supply problem and the new plans are outlined in an article in this issue.

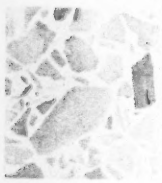
COSTS OF KNICKERBOCKER VILLAGE, the large RFC-financed housing project in New York City, have been broken down in an interesting classification by the U. S. Bureau of Labor Statistics, a useful picture of what one type of low-rent housing costs is the result.

Coming Articles:

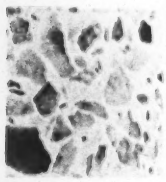
CURRENT ADVANCES in sewage disposal, a page of timely notes on design, operation and research in this rapidly developing branch of sanitary engineering, will be inaugurated as a regular monthly feature beginning with the issue of Dec. 26. Thoughtful selection and interpretation of data relating to stream pollution, sewage and trade-wastes treatment will be the aim of each installment.

Fine terrazzo ... graphed in f... in St. Francis ... Paducah, Ken... Atlas White ... Keno Rosa C... contractor. Th... architect. Both ... actual-size, tru... samples below ... and pigment co... lar to those us...

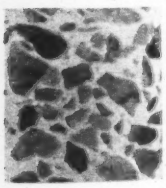
Botticino marble ... white portland cement



Yellow Verona marble with white portland cement and yellow pigment



Verdolite marble ... white portland cement and green pigment



Red Verona marble ... white portland cement and red pigment

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ATLAS WHITE PORTLAND CEMENT

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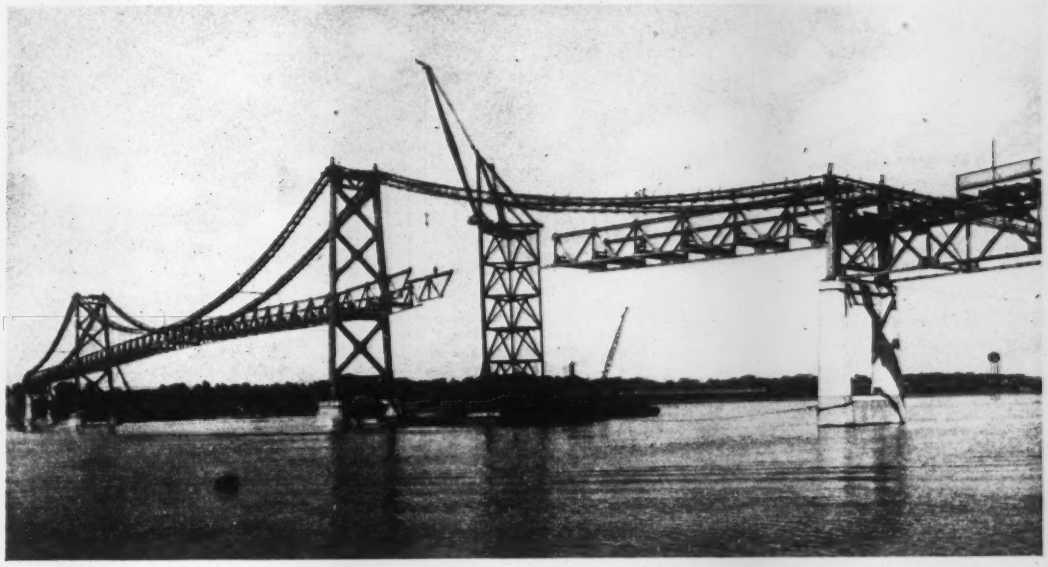
ENGINEERING NEWS-RECORD

December 19, 1935



FLOATING TOWER DERRICK hanging stiffening trusses and floor steel on the Davenport Bridge. The same equipment erected the bridge towers.

Building the Davenport Bridge Across the Mississippi

New toll crossing over pool above Rock Island Dam includes a 740-ft.-span suspension bridge and a series of continuous-truss spans—Suspension-bridge towers and stiffening trusses erected by floating-tower derrick

THE NEW Mississippi River crossing of the Davenport (Ia.) Bridge Commission, opened to traffic on Nov. 18, has constituted one of the substantial bridge-building operations of the year. Spanning the pool formed by the navigation dam at Rock Island three miles downstream, the bridge is divided into two parts by a narrow island that is submerged at times of high water. North of this island the structure is a twisted-wire-strand cable suspension bridge with a 740-ft. main span. South of the island are two three-span continuous-truss units supported on seven piers, each span being 222 ft. long. The approaches are of rolled-beam construction, using continuous spans.

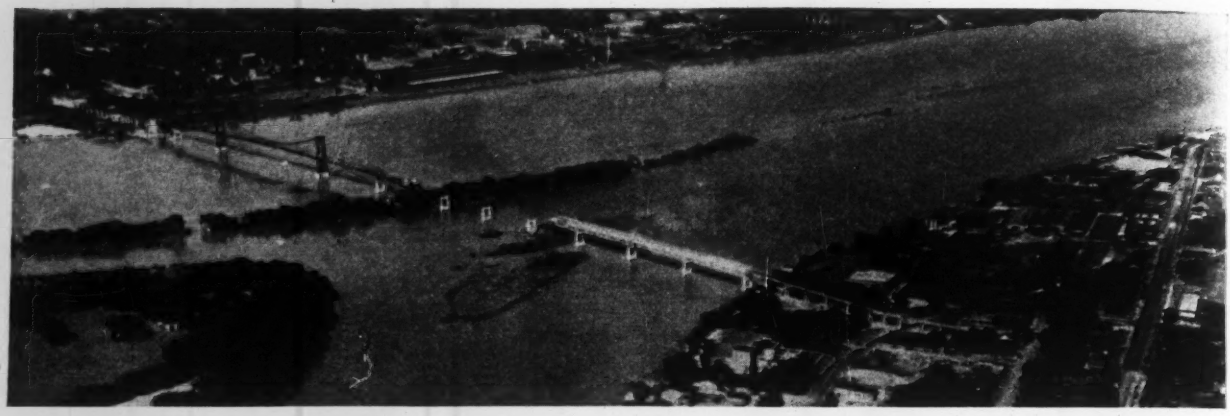
Paradoxically, neither end of the Davenport Bridge is in Davenport, the south approach coming to grade in Moline, Ill., and the north approach in Bettendorf, Ia., the Mississippi at this point running east and west. Davenport is about 3 miles downstream from Bettendorf. Projected as a toll bridge, the new crossing will accommodate two lanes of traffic on a 20-ft. roadway, which is widened in the center of the structure at the south end of the suspension bridge to accommodate the toll-collection facilities. The bridge was financed by the Public Works Administration, the total cost being about $1,500,000.

Design features

In design the bridge has several notable features. The rock foundations, which were available at relatively shallow depths, permitted the use of continuous-truss spans over the Moline Pool and continuous rolled-beam spans on each approach. Where three-span continuous-beam units are used on the approaches, they are fixed at the two intermediate piers, which are designed to deflect under temperature change.

In the bridge superstructure, considerable economy was effected by a liberal use of silicon steel and of wide-flanged rolled beams. Careful consideration was given to the design of the structural details. Lacing bars are practically eliminated in the structure, being employed only in the heavy cross-bracing of the main towers and in the columns of the cable bents.

Since the design of the towers of a



AN ISLAND marks the dividing point between the suspension bridge over the Bettendorf navigation channel and the continuous-truss bridge over the Moline Pool in the Mississippi River.

suspension bridge largely determines its appearance, particular attention was given to the proportions of these towers. They are of the fixed-base, flexible type, the tower legs consisting of rolled sections and plates, to form three vertical cells, the outer cells being varied in dimension to give the tower a curved outline when the bridge is seen in elevation. The cables for the suspended spans are of prestressed twisted-wire rope, each cable consisting of 31 strands of 1½-in. diameter and six strands of 1-in. diameter, wrapped to circular form using aluminum and zinc fillers in the interstices of the outside strands.

Construction of the bridge involved some noteworthy, although not unprecedented, practices. For example, the eleven river piers were sunk by the pneumatic method, although the water has a maximum depth of only 20 ft. and the bottom is rock. The towers and stiffening trusses of the suspension bridge were erected by a floating tower derrick more than 100 ft. high, and special jacking operations were required in landing the various spans of the continuous trusses.

Foundations

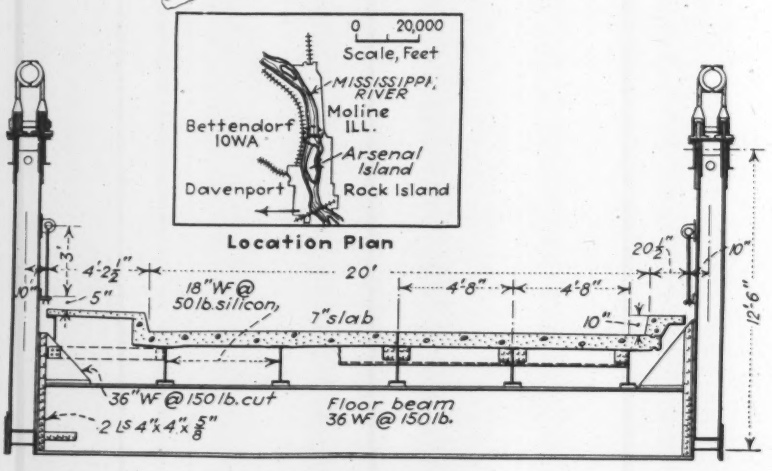
A total of 45 foundations, all carried to rock, were required for the bridge, eleven of which, including the south anchorage of the suspension bridge, are in the river. Rock is comparatively near the surface of both approaches, which permitted the footings to be built in sheeted holes of 6- to 16-ft. depth. In the riverbed the rock dips down but seldom to a depth greater than 21 ft. below water level. It is overlaid with a shallow stratum of sand. Since the rock surface was very irregular, the contractor elected to put the river piers and anchorage down by the pneumatic method in the belief that by this means better construction could be accomplished in less time than by using open cofferdams. The procedure followed is worth noting.

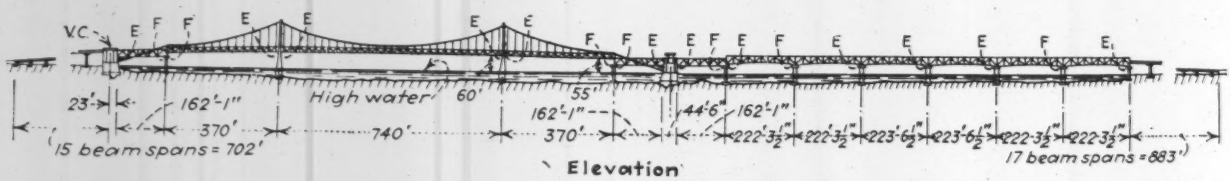
All cutting-edge sections, including about 6 ft. of steel caisson shell, were fabricated by welding in St. Louis. Launched upside down, they were towed the 250 miles up the Mississippi River to the bridge site; several of the smaller pier sections were lashed together to serve as a pontoon barge for the large cutting-edge section of the anchorage caisson. Arriving at the bridge site, the sections were turned over and raised to a fitting-up barge where the timber framework and sheathing of the top section were added. The barge was then towed to the pier location, where the caisson was picked up and lowered by a special handling device.

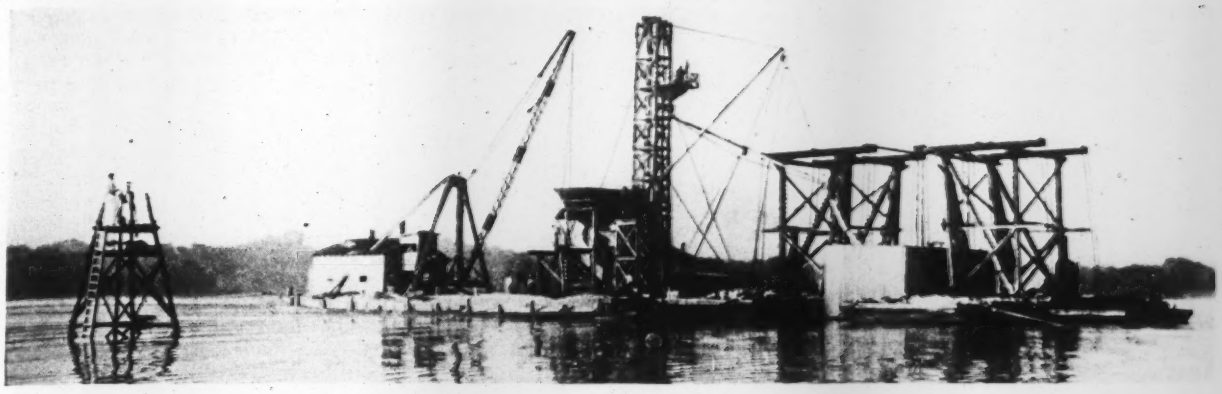
This caisson-lowering device consisted of two barges equipped with timber frames and steam hoisting engines. On top of each frame were two pairs of channels supporting sheaves for the hoist lines, which terminated in balance beams and hooks for engaging the four corners of the caisson. Each equalizing beam was supported by two seven-part lines. The barges were counterbalanced to pick 40 tons per barge. The anchorage caisson, after it was fitted with timber coffer, was removed from the pontoons by filling the latter with water, sinking them and allowing the caisson to float.

The caisson lowering device controlled the caisson until it had been landed on the river bed. Just before

ELEVATION, section and location plan of Davenport Bridge.

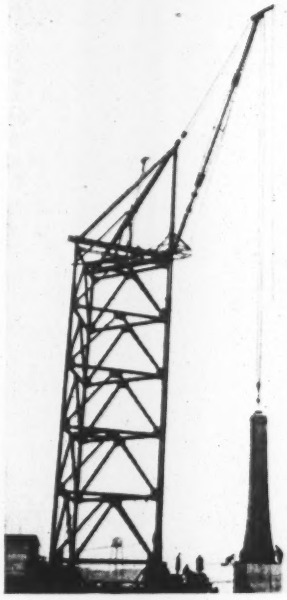






Lowering a caisson with the special headframe device.

DAVENPORT SUSPENSION BRIDGE

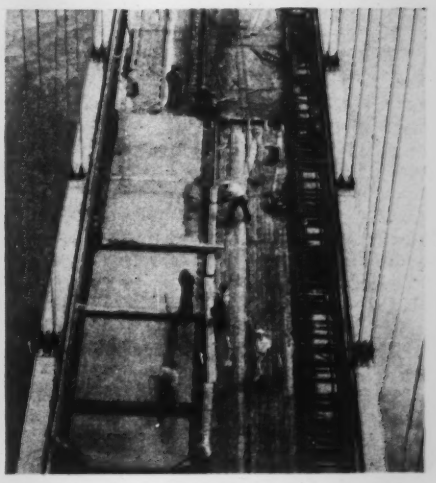


Setting tower steel.

Assembling the cable-construction equipment.

FROM PIERS TO PAVING

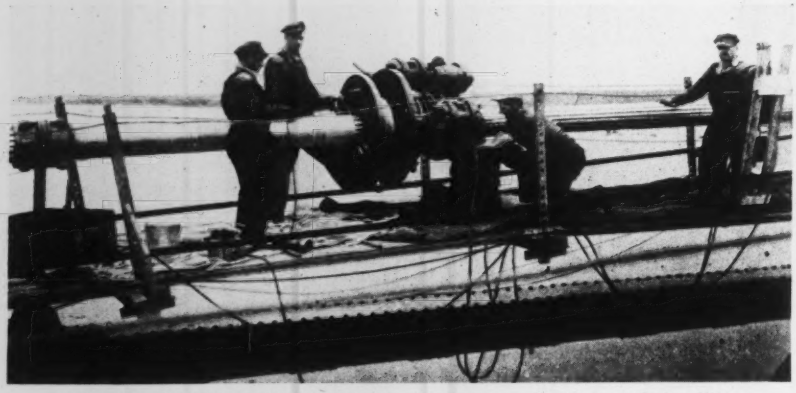
Installing floor steel in toll-collecting area over the island.



Paving being placed in checkerboard fashion.

landing the caisson was located accurately a few inches from the bottom, a transit on a triangulation tower in the river being used. At most, this adjusting to accurate location took an hour, since the caisson was under positive control by the lowering device at all times. The caisson being accurately in position, the four supporting lines were suddenly released, dropping the caisson on the bottom.

The lowering device was also used to lift the timber top sections from the river piers after they had been completed and to transfer them to the other caissons to be used over again. This method of handling the caissons proved to be both speedy and accurate.



CABLE WRAPPING beginning at center of main span. Aluminum and zinc fillers bring cable to a circular section before wire wrapping is applied.

Each pier caisson was equipped with one man lock and two material locks; a total of six locks was used on the river anchorage caisson. Excavation was partly by bucket and partly by blow pipe. A floating concrete-mixing plant was used. Foundation progress adhered closely to a predetermined schedule. Only one cavity was found in the limestone rock beneath the piers, and this cavity had been detected in the borings. The cavity was filled with concrete before sealing the caisson.

Suspension-bridge erection

The most spectacular part of the job of building the Davenport Bridge was the erection of the suspension-bridge towers and stiffening trusses by a floating tower derrick capable of setting steel nearly 200 ft. above the water level. Such equipment has been used once before—in 1931, on the bridge over the Ohio River at Maysville, Ky.—but in that case only the towers and not the stiffening trusses were erected by the tower derrick.

The Davenport tower derrick was triangular in plan, 33 ft. on a side and 112 ft. high above the two steel barges on which it rested. On top of the tower a 35-ton steel stiff-leg derrick with 90-ft. boom was erected. The barges, 30 ft. wide by 130 ft. long by 7½ ft. deep, had an average draft of 4.25 ft. with a 30-ton load in the derrick falls. In designing the equipment extreme care was exercised in determining the draft and trim of the boat under all possible sizes and positions of loads. The base of the tower consisted of two plate girders spanning from barge to barge and resting on two 36-in. rolled beams placed lengthwise of each barge. One plate girder supported the front leg of the tower, while the other girder carried the two rear legs. Two counterweights of 50 and 85 tons, respectively, were used on each barge.

The derrick was operated by a three-drum steam hoisting engine and a swing engine. A six-part falls and a thirteen-part topping lift were used.

The Davenport suspension bridge has a main span of 740 ft. and suspended side spans of 370 ft. Between the cable bent pier and the anchorage on each side is a 162-ft. deck span. Erection began with the towers, the steel being brought out from a Bettendorf storage yard on barges. With the tower complete, the 162-ft. Bettendorf approach-span trusses, weighing 41 tons, were set without falsework, using a locomotive crane on the shore end and the tower derrick for the end at the cable bent. The trusses were spliced in the air while hanging from the hoist lines. A similar truss on the east end of the bridge was spliced on barges and erected entirely by the tower derrick.

Work was shut down Dec. 21 because of heavy ice in the river and was not resumed until March 20, although most of the Bettendorf approach work was completed during the winter months. When work was resumed in the spring, everything was ready for cable construction. The hauling-rope supports on top of the main towers had been placed previously, and the first operations, therefore, consisted of installing the hauling rope and erecting the footwalks and storm cables. After the guide strands in the cable were placed and adjusted and the pull-back cables were attached to the towers and cable bents, erection of the strands of the cables was carried out in the usual way, the hauling rope pulling a strand in one cable from north to south, and while that strand was being pinned off pulling another strand in the same direction in the other cable. Stringing of 37 strands in both cables, after the two guide strands had been set, required only six working days. Stringing of twelve strands, six in each cable, took on an average of 220 minutes in the morning of each day. A small gang was used in the afternoon to make the preliminary adjustments, while the final adjustment of each layer of strands was made between midnight and 5 a.m., to allow stringing the next layer the following morning. Cable-band erection required two days, and suspenders were hung as necessary by the floating tower derrick at the time the stiffening trusses were erected.

Hanging the stiffening trusses, which were fabricated in sections 50 to 80 ft. long, weighing 8 to 13 tons, was begun at the center of the main span. After sections had been hung from the twelve center panels of the 30-panel main span, the tower derrick was moved to the side spans where it erected the six end panels. The ends of the center span were then filled in, followed by completion of the side spans. This order of erection kept the main tower deflections within a limit of 12 in. in either direction.

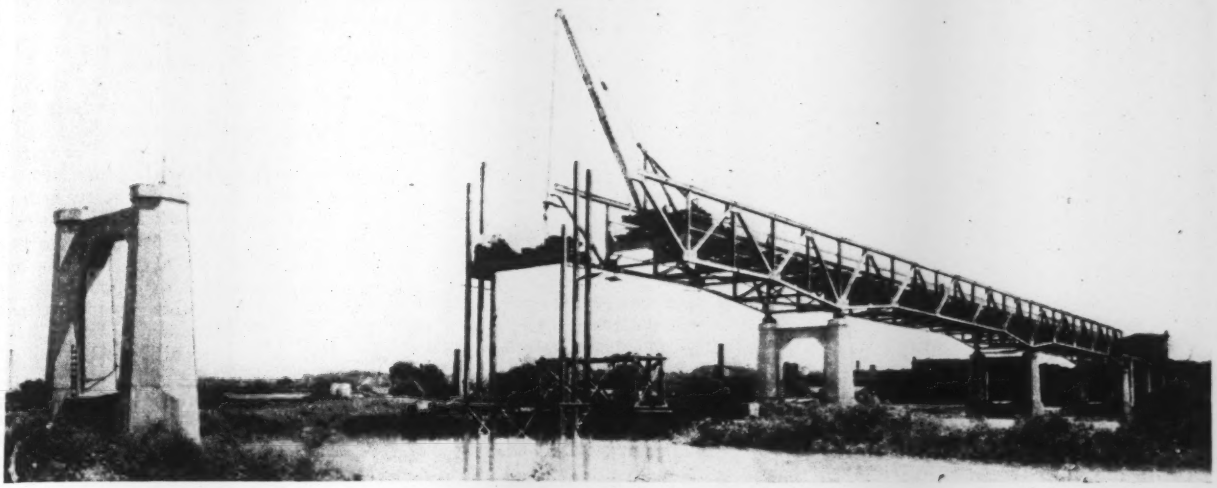
The majority of the floorbeams were erected with the stiffening trusses. The stringers, sidewalk framing and railing were erected as a separate operation of the floating derrick, working continuously from one end of the suspended span to the other.

The program for steel erection and paving of the suspension bridge was determined from a model constructed by the engineering department of the contractor. The program worked out from the model was used without change in the field. Model studies gave figures on tower deflections, cable elevations and cable distortions for any loading and with sufficient accuracy for planning the field work, and the use of a model saved many hours of laborious computation.

All rivets except the top-chord splices were driven in the suspension bridge before the paving was laid. Pavement concrete was poured in checkerboard pattern in blocks 100 ft. long and 10 ft. wide (one-half roadway width), to keep tower deflections and distortion of the stiffening trusses within reasonable limits. The concrete was distributed from the Bettendorf mixing plant in trains of four ½-yd. hopper cars pulled by a narrow-gage gasoline locomotive on tracks laid in the sidewalk area. Following completion of the paving, the cables were wrapped and the footwalks removed, completing the suspension bridge.

Continuous-truss erection

The two three-span continuous-truss units of half-through construction, com-



CANTILEVER ERECTION of continuous-truss spans using a falsework bent of six steel H-piles equipped with jacks to lift the span for landing on pier ahead.

prising the south half of the bridge called for some special erection procedure, since it was desired to erect the spans by the cantilever method. The continuous units were designed as a series of simple spans for steel dead loads and later made continuous for the roadway slab and live load. Since there was little clearance between the ends of the trusses over the piers, it was not possible to anchor the trusses sufficiently to erect by the cantilever method in a position higher than the final position. As an alternative, adjacent spans were tied together for tension only by means of straps along the web of the top chords, and the projecting cantilever ends were allowed to droop until they could be picked up at the falsework bents on jacks. Erection began at the south end, the steel being unloaded under the Moline approach, lifted to small trucks on the deck and hauled out to the erection traveler. This traveler was made only 12 ft. wide, to allow the steel to be delivered alongside by standard-gage gasoline dinkey and trailers. The traveler consisted of a small stiff-leg derrick mounted on a steel platform equipped with flanged wheels for operating on two 85-lb. rails laid directly on the stringer flanges. It carried a two-drum gasoline hoist.

Falsework consisted of towers of six steel H-piles braced at about water level by a steel-frame cage about 12 ft. deep and above this by a crossbracing of wire cable extending to the jacking girders upon which the trusses were supported. The pile flanges were punched with a series of holes, to permit the girders to be attached at a convenient level. The bracing cage and piles were moved from span to span on a barge. Lifted from the barge by the deck traveler, the piles were set in the frame and lowered to the bottom of the river, where they were driven as far as possible into the rock. A 2,000-lb. drop hammer was used, handled by an air hoist, since there was no whip line on the traveler engine; the swinging hammer leads were handled by the main falls.

Two complete six-pole bents were provided, since three falsework supports were required under the first span. The procedure was to set falsework bents under the second and fourth panel of the south span, erect these four panels, remove the bent under panel point 2 and transfer it to panel point 6, after which the span was completed by cantilevering to the next pier. With this span in place as a beginning anchor, the other five spans were each cantilevered for six panels (134 ft.), landed on a six-pile bent and again cantilevered 88 ft. to the next pier. One 250-ton hydraulic jack in combination with wood-camber blocking was used under each bottom chord on the falsework jacking beam. Depending on conditions, it was necessary to jack up from 10 to 14 in., after landing on the falsework, to get high enough to land on the pier ahead. The bottom-chord section adjacent to the piers had to be increased somewhat in section since the cantilever stress was higher than the design stress. Also, one truss diagonal over the falsework bent was temporarily strutted to increase its stiffness and hence its capacity.

The last four 220-ft. spans, weighing about 190 tons, were erected in an average of 60 working hours each (minimum, 55 hours), this time including erection and removal of the falsework bents. The maximum load handled by the traveled was the pile cage, which weighed 6½ tons at about 45-ft. reach. The piles were pulled with the traveler main falls aided by compressed-air jetting where the penetration was from 10 to 12 ft. in mud and clay overlying rock.

The work was done in two six-hour shifts per day, five days per week. All men except the supervisory forces were employed through the National Re-employment Service at Davenport and Rock Island.

The bridge was designed and its fabrication and erection were supervised by Modjeski, Masters & Case, Inc., consulting engineers, Philadelphia, with W. C. Gorman as resident engineer.

The substructure contract was carried out by the Kansas City Bridge Co., I. E. Hayes, superintendent. Fabrication and erection of the superstructure were handled by McClintic-Marshall Corp., for whom H. E. Crider is manager of erection, Western District; L. L. Martin was resident engineer.

Electric Rate Survey Standardizes Definitions

Confusion in terminology used in electric rate schedules and contracts throughout the United States has led to the preparation of a glossary of 161 definitions by engineers and accountants of the Electric Rate Survey now being conducted under the direction of the Federal Power Commission. This is being checked by a selected group of utility and regulatory officials, engineers and other experts for criticism before publication.

Need for a glossary developed during the study of reports from operating utilities. It was found that that there is no community of understanding regarding numerous terms used in rate schedules and contracts, although the amount of the customer's bill was often materially affected by the interpretation of a term to which no definite meaning could be assigned. Although the glossary was prepared primarily for its own use, the Electric Rate Survey believes that publication will help to eliminate the confusion prevailing in rate and power terminology.

Cast-in-Place Short Piles Show High Test Results

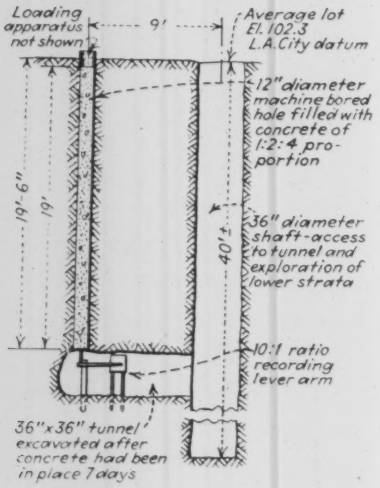
Small differential settlement between footings of structures founded on short concrete piles recommends their use in tender soils

By Frederick J. Converse
Labarre & Converse, Foundation Engineers, Los Angeles, Calif.

THE extremely loose and friable soils found in many portions of the Los Angeles basin have made the problem of economical foundations for low and medium-height buildings a vexatious one, especially since lateral-force requirements have been introduced into the building codes. A number of years ago R. V. Labarre, of Los Angeles, Calif., suggested that the use of short cast-in-place concrete posts might solve the problem for some conditions. The tests described below indicate that this type of foundation has many advantages in weak soils.

One of the main objections of engineers to the use of cast-in-place piles has been the difficulty of insuring perfect piles without pockets or slugs of caved material. This is a real danger, and whenever such piles are used proper precautions must be taken to make sure that the piles are sound. The method to be used will depend upon the character of the soil and the depth of the hole. In the tests described below the holes were excavated by means of a hand soil auger. The soil stood well with no tendency to cave, and the depths were not great, so that no precautions were used in placing beyond slow careful pouring of a plastic concrete mix and careful rodding from the surface. Excavation of some of the posts after the tests were completed showed them to be in perfect condition.

FIG. 1—TEST ARRANGEMENT for determining settlement of a cast-in-place concrete pile under load.


Loading apparatus not shown
9'
Average lot El. 102.3 L.A.City datum
12" diameter machine bored hole filled with concrete of 1:2:4 proportion
19'-6"
19'
40'±
36" diameter shaft-access to tunnel and exploration of lower strata
10:1 ratio recording lever arm
36"x36" tunnel excavated after concrete had been in place 7 days

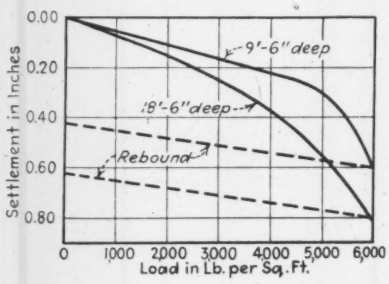
First test

A 12-in.-diameter concrete post 19 ft. deep was cast in place in loosely compacted sandy loam. To eliminate the effect of end bearing and to measure the movement of the bottom of the post, a 3-ft. shaft was bored at a distance of 9 ft. from the post, and a tunnel extended horizontally from it to a point beneath the post, as shown in Fig. 1. Seven days after pouring the concrete, load was applied and gradually increased to 43,000 lb. over a period of one hour. Settlement increased with load to a maximum of $\frac{5}{32}$ in. During the second hour the load was gradually increased to 70,000 lb. and then allowed to remain in place for six hours. No further settlement occurred.

A comparison between the bearing capacity and settlement of the post and that of spread footings may be made by considering the curves in Fig. 2, which show the results of load tests on 4-sq.ft. flat plates at depths of 9½ and 18½ ft. Compared with a settlement of 0.03 in. under a total load of 70,000 lb. obtained with the post, the flat plates settled 0.14 and 0.24 in. respectively under 2,500 lb. per sq.ft. load, and 0.22 and 0.37 in. respectively under 4,000 lb. per sq.ft. load. The concrete required for the post is 14.9 cu.ft., while that for a spread footing 1 ft. thick, carrying 70,000 lb., is 17 cu.ft. at 4,500 lb. per sq.ft., and 28 cu.ft. at 2,500 lb. per sq.ft.

In considering the cost of the two types of foundation the extra steel reinforcing and excavation required for the spread footing must also be taken into account.

FIG. 2—SETTLEMENT of 4-sq.ft. plates at two depths under increasing load.



Second test

This test was made in sandy loam of recent age, deposited by the flood waters of the Los Angeles River. The surface material was very loosely compacted, becoming somewhat denser below a depth of 4 ft. Load tests indicated a safe bearing value for spread footings less than 4 ft. deep of only 1,500 lb. per sq.ft., and for deeper footings 2,500 lb. per sq.ft.

A 12-in. cast-in-place concrete post 10¾ ft. long in this material carried 24,000 lb. with a settlement of 0.10 in., and 30,000 lb. with 0.15-in. settlement. The load-settlement curves for the post and for a 4-sq.ft. plate located 6 ft. 5 in. deep are shown in Fig. 3.

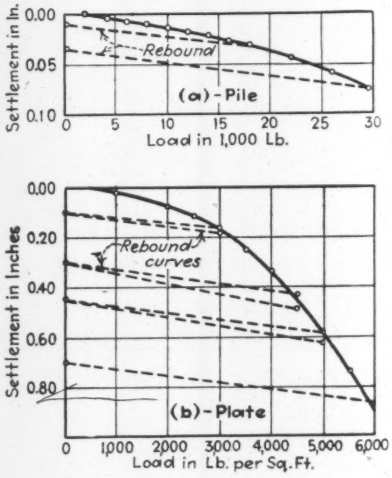
In this case a post containing 8.4 cu.ft. of concrete and requiring 8.4 cu.ft. of excavation carried as much load as a 9.6-sq.ft. spread footing located at least 4 ft. below the surface. Although there was in this case, as in the previous one, a saving in material, excavation and formwork, the most important practical feature was the small settlement that occurred. This was especially desirable at the site in question, because the soil consists of thinly bedded layers of irregularly deposited material ranging from soft clay to firm gravel, and the design of foundations to produce uniform settlement of spread footings on such material is complex and unsatisfactory.

Third test

Another interesting pair of tests was made in loosely compacted and irregularly bedded material varying from very fine sand to sandy loam, with streaks of silty loam. The field-compression test was made on a 4-sq.ft. circular plate bedded in sandy loam at a depth of 8 ft. below the surface.

A concrete test post 12 in. in diameter

FIG. 3—SETTLEMENT of cast-in-place concrete pile contrasted with settlement of 4-sq.ft. plate in similar soil in second test.



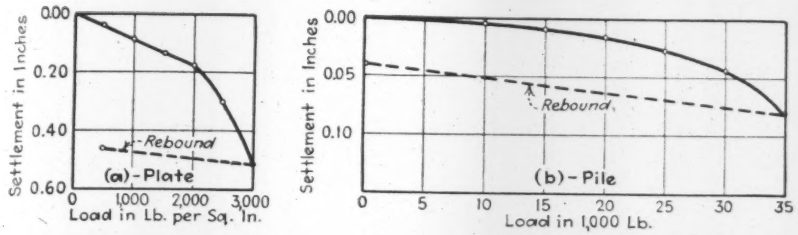


FIG. 4—COMPARATIVE SETTLEMENT of concrete pile and 4-sq.ft. plate in third test.

and 20 ft. deep was cast in place near the location of the compression test. Loading was started the day after pouring the quick-setting concrete, and was completed in 28 hours. The load-settlement curve for this test is shown in Fig. 4*b*. The post was allowed to come to rest after each increment of load, except for the final load or 40,000 lb. Settement under this load was still progressing at the rate of 0.001 in. per hour after fifteen hours, and time did not permit continuing the test to rest.

The break in the load-settlement curve for the flat plate occurs at about 2,000 lb. per sq.ft. Assuming 2,000 lb. per sq.ft. as the allowable unit load on the plate, 15 sq.ft. is required to support 30,000 lb., providing the bottom of the footing is at a depth of at least 8 ft. below the surface. The settlement of the spread footing will be at least 0.2 in. under this load, and probably more because of the size effect. An increase of 20 per cent in load causes an additional settlement of 0.1 in. for the spread footing, but only 0.04 in. for the post. Stresses in a structure due to differential settlement of footings will therefore be less for post foundations than for spread footings.

Shear tests were made on the soil, and the bearing values of both the post and the plate were calculated from them. The shearing strength of the undisturbed material immediately below the plate was 550 lb. per sq.ft., and that 2 ft. deeper was 700 lb. per sq.ft. Using an average value of 625 lb. per sq.ft., calculations based on the theory of elasticity indicated that the beginning of shear failure should occur at a unit vertical load on the plate of $\pi \times 625$, or 1,960 lb. per sq.ft., and that the maximum average unit pressure on the plate should be $(\pi + 2) \times 625$, or 3,200 lb. per sq.ft.

The break in the load-settlement curve is close to the calculated value for the beginning of shear failure, and at an average unit pressure of 3,200 lb. per sq.ft. the settlement of the plate is very rapid. There is thus a close agreement in this case between results obtained by the ideal elastic theory and those obtained by field tests.

For the case of the concrete post, penetration occurs when there is slip between the soil and the post or by shear in the soil surrounding the post when the bond between the concrete and the soil is greater than the shear value of the soil.

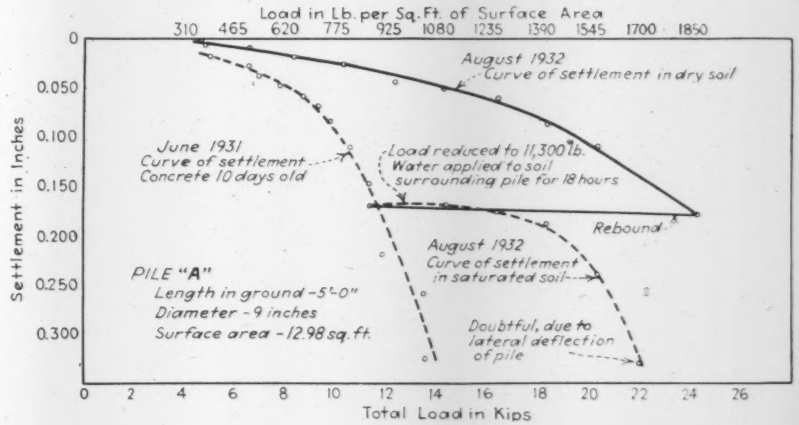
Multiplying the nominal cylindrical area of the post by the average shearing value of the soil along the full length (575 lb. per sq.ft.) gives a total load at the beginning of shear of 36,000 lb. This value is high compared with the test (see Fig. 4*b*), as would be expected, since the test was made only one day after casting the concrete, and there was not sufficient time for thorough bond to develop. End bearing will also account for a small amount of load, but not to exceed 1,000 lb.

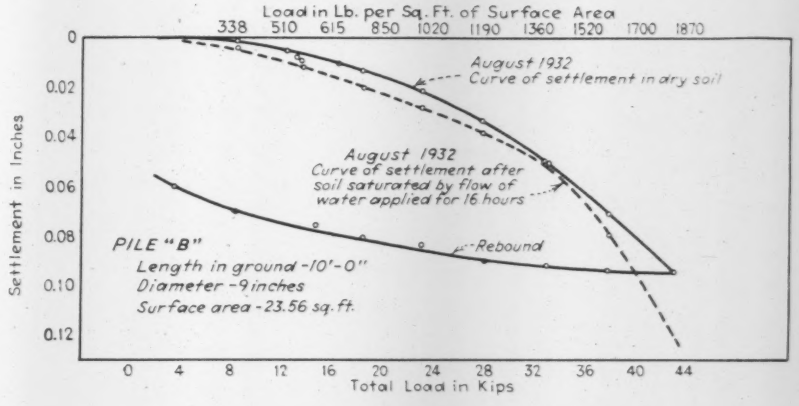
Fourth test

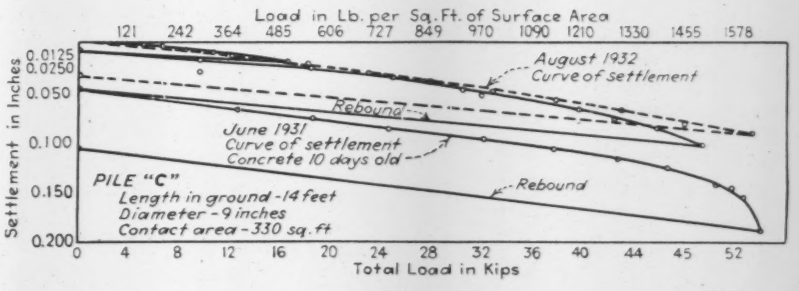
The increase in bearing capacity of a pile with time is a recognized fact among foundation men. In 1931 and 1932 a series of tests was made at the California Institute of Technology, which indicates how important this element may be.

In these tests three concrete piles 9 in. in diameter and 5, 10 and 14 ft. respectively in depth, were cast in soil that varied progressively from soft loam at the surface to clay loam at a depth of 2¼ ft. to sandy clay at 6 ft., to loosely

FIG. 5—LOAD SETTLEMENT in terms of surface area of concrete piles under different conditions of soil saturation.







compacted sand containing gravel below 8 ft.

Ten days after pouring concrete in June, 1931, pile A (5 ft. long) was tested, and fifteen months later it was again loaded. A comparison of the two load-settlement curves is given in Fig. 5A.

In the 1932 test, after a total load of 24,000 lb. had been applied and settlement had ceased, the load was reduced to 11,300 lb. and held constant at this value for eighteen hours. During all of this eighteen-hour period a pond of water was maintained about the top of the pile. Load was then re-applied in increments of 2,000 lb. up to a total of 24,000 lb. At this pressure the loading post shifted out of plumb, and lateral pressure caused considerable deflection. The test was therefore stopped, and the post was dug out, disclosing a break at the mid-point. The moisture content of the soil in the immediate vicinity of the pile was undoubtedly high in the 1931 tests due to water from the fresh concrete. In 1932 it varied from 4.5 to 6.9 per cent before ponding, and from 9.9 to 15.5 per cent after ponding.

Pile C (Fig. 5), 14 ft. long, was also tested in both 1931 and 1932. In the 1931 tests the ultimate capacity of the pile was reached, but in the 1932 tests there was no indication of approach to the ultimate, although the same total load was applied. Lack of loading capacity prevented carrying the test to ultimate failure. The results are plotted graphically in Fig. 5C.

Pile B (Fig. 5B), 10 ft long, was not tested in 1931 but was tested both dry and wet in 1932. In the curves the full line indicates the results of the test with a moisture condition similar to that of test A in 1932. After this test the load was completely removed, and water was kept ponded about the top for sixteen hours, after which the load was again applied. The second loading is indicated by a dash line in Fig. 5B, starting from the same origin as the first curve.

These tests bring out strikingly the increase in bearing capacity with time. The lack of serious reduction in capacity due to increase in moisture, after thorough bond between the concrete and the soil has been established, is also worthy of note.

Conclusion

Considering all of the tests here presented, one important fact common to all of them stands out—namely, the settlements under load are small for cast-in-place concrete posts compared to those of spread footings. The stability of the posts and the small differential settlements between footings of a structure founded on such posts strongly recommends their use in tender soils of the type described.

The first of the above tests was made by R. V. Labarre several years ago as a demonstration to the Los Angeles building department. The last series was made in cooperation with the California Institute of Technology. The other tests were made by Labarre & Converse as opportunity arose in the regular course of their foundation investigations.

Coefficients for Beam Deflections Under Various Loading Arrangements

By Robins Fleming
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THE SERIES of tables presented in this article will permit the vertical deflections of beams to be determined with sufficient accuracy for all practical purposes. The fundamental equation upon which deflections are based is $\frac{d^2y}{dx^2} = \frac{M}{EI}$. In this article E and I are considered constants. The deflection coefficients, F, given in the tables, are for Pl^3/EI in case of a load P concentrated at specified points and for Wl^3/EI at specified points in case of a load W uniformly distributed over the length of the beam. Since E is assumed in pounds, W and P must be considered in pounds. If I and l are considered in inches, the deflection of the beam will be in inches. The distance from the left end of the beam to the point where deflection is given is considered as kl. The distance from mid-span to point of maximum deflection is considered as $k_1 l$.

Only a few comments relative to the tables are necessary. Maxwell's principle of reciprocity of displacements renders valuable service in determining beam deflections. It states "If A and B are two points on a beam (or any elastic structure), the deflection at A caused by a given load at B is equal to the deflection at B caused by the same load at A." Thus in the cantilever beam (Case II) the deflection at the free end from a load at any point, say at $k = 0.3$, is the same as the deflection at $k = 0.3$ from the same load placed at the free end. The maximum deflection is always at the free end, of course, whatever the type of loading.

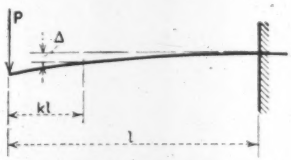
In Case IV and VIII, it is obvious that the deflection from a single load is the same when placed at the same distance from either end of the beam. Thus, the deflection of a load placed at $k = 0.3$ is the same as if placed at $k = 0.7$. It is observed that the point of maximum deflection from a single load is always between the load and the center of the beam, and the distance from the center (k, in the tables) is small. In Case IV, if the load were placed adjoining a support, the point of maximum deflection is but 0.0772 the length of the beam from its center. The difference between the deflections at these points is thus slight. The more loads on the beams the less is the difference. For a series of concentrated loads the total deflection at any point is the sum of the deflections at that point from each separate load. The total deflection at the center can easily be obtained, and more than this will seldom be wanted.

For Case IV Goodman ("Mechanics of Engineering") gives a table of coefficients for a load placed at consecutive tenth-points of the span, with the deflection at every tenth-point. Ellis in his "Theory of Framed Structures" gives a similar table for points varying by 0.05 l. Ellis also includes in his table the location of the point of maximum deflection for each load.

The deflection from partial or discontinuous uniform loading can be determined by considering the load on the length over which it extends concentrated at the tenth-points within that length; that is, at each tenth-point will be concentrated the uniform load on 1/20th the span length on each side of the point. The deflections at mid-span from each concentrated load can be taken from the tables for Case IV or VIII. Their sum will be the total deflection at mid-span, which will be a close approximation to what is wanted. Fortunately, a uniform load usually extends the entire length of the beam.

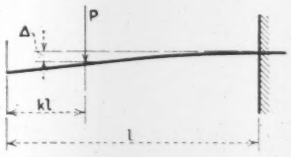
For Case IX, since the beam is simply supported at one end and fixed at the other, the deflection from a single load is not the same when the load is placed the same distance from the other end. Neither is the relation between maximum deflection and deflection at mid-span as close as in Cases IV and VIII. However, the point of maximum deflection always comes within the middle third of the beam length. The maximum deflection from a single load occurs when the load is placed at a distance 0.414 l from the supported end and amounts to 0.0098 Pl^3/EI. In Case VII, for a uniform load, the maximum deflection is at a distance 0.4215 l from the free end and has a value of 0.00541 Wl^3/EI.

In the foregoing no account is taken of the deflection due to shear. It seldom need be considered except in short deep beams heavily loaded. In a 24-in. beam 10 ft. long supported at the ends, the deflection due to shear is from 15 to 25 per cent of that due to bending. In a 12-in. beam of the same length the shearing deflection is from 4 to 6 per cent of the bending deflection. Sometimes, as in the supports of turbo-generators, practically no deflection is permissible. These, however, are special cases and should be treated accordingly.



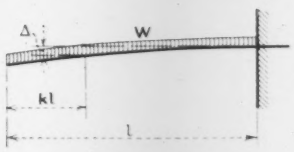
k	F
0.0	0.3333
0.1	0.2835
0.2	0.2346
0.3	0.1878
0.4	0.1440
0.5	0.1041
0.6	0.0693
0.7	0.0405
0.8	0.0186
0.9	0.0048
1.0	0.0000

Case I - Cantilever with load at end



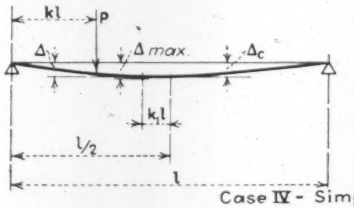
k	F
0.0	0.3333
0.1	0.2430
0.2	0.1706
0.3	0.1143
0.4	0.0720
0.5	0.0416
0.6	0.0213
0.7	0.0090
0.8	0.0026
0.9	0.0003
1.0	0.0000

Case II - Cantilever with load at tenth point



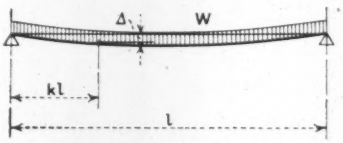
k	F
0.0	0.1250
0.1	0.1083
0.2	0.0917
0.3	0.0753
0.4	0.0593
0.5	0.0442
0.6	0.0303
0.7	0.0182
0.8	0.0087
0.9	0.0023
1.0	0.0000

Case III - Cantilever with uniform load



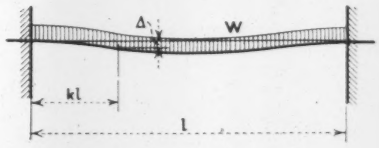
k	VALUES OF F			k_1
	Under load	At mid-span	Maximum	
0.1	0.0026	0.0061	0.0063	0.0745
0.2	0.0085	0.0118	0.0120	0.0657
0.3	0.0147	0.0165	0.0167	0.0508
0.4	0.0192	0.0196	0.0197	0.0292
0.5	0.0208	0.0208	0.0208	0.0000

Case IV - Simple Beam with load at specified points



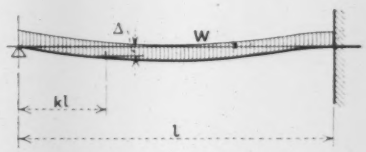
k	F
0.1	0.0004
0.2	0.0077
0.3	0.0105
0.4	0.0124
0.5	0.0130

Case V - Simple beam with uniform load



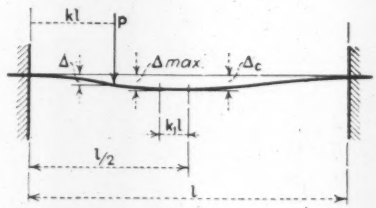
k	F
0.1	0.0003
0.2	0.0010
0.3	0.0018
0.4	0.0024
0.5	0.0026

Case VI - Beam fixed at both ends, with uniform load



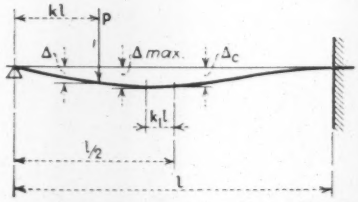
k	F
0.1	0.0020
0.2	0.0037
0.3	0.0049
0.4	0.0054
0.5	0.0052
0.6	0.0044
0.7	0.0031
0.8	0.0017
0.9	0.0005

Case VII - Beam Fixed at one end, with uniform load



k	VALUES OF F			k_1
	Under load	At mid-span	Maximum	
0.1	0.0002	0.0005	0.0006	0.1428
0.2	0.0013	0.0018	0.0020	0.1153
0.3	0.0030	0.0033	0.0035	0.0833
0.4	0.0046	0.0046	0.0047	0.0454
0.5	0.0052	0.0052	0.0052	0.0000

Case VIII - Beam fixed at both ends, with load at specified points



k	VALUES OF F			k_1
	Under load	At mid-span	Maximum	
0.1	0.0018	0.0030	0.0036	0.1623
0.2	0.0054	0.0058	0.0067	0.1487
0.3	0.0084	0.0079	0.0089	0.1289
0.4	0.0097	0.0091	0.0097	0.0916
0.5	0.0091	0.0091	0.0093	0.0528
0.6	0.0069	0.0076	0.0076	0.0197
0.7	0.0040	0.0053	0.0053	0.0091
0.8	0.0016	0.0028	0.0028	0.0345
0.9	0.0002	0.0008	0.0008	0.0570

Case IX - Beam fixed at one end, with load at specified points

DEFLECTION COEFFICIENTS *F* for various types of beams under both concentrated and uniform loads.

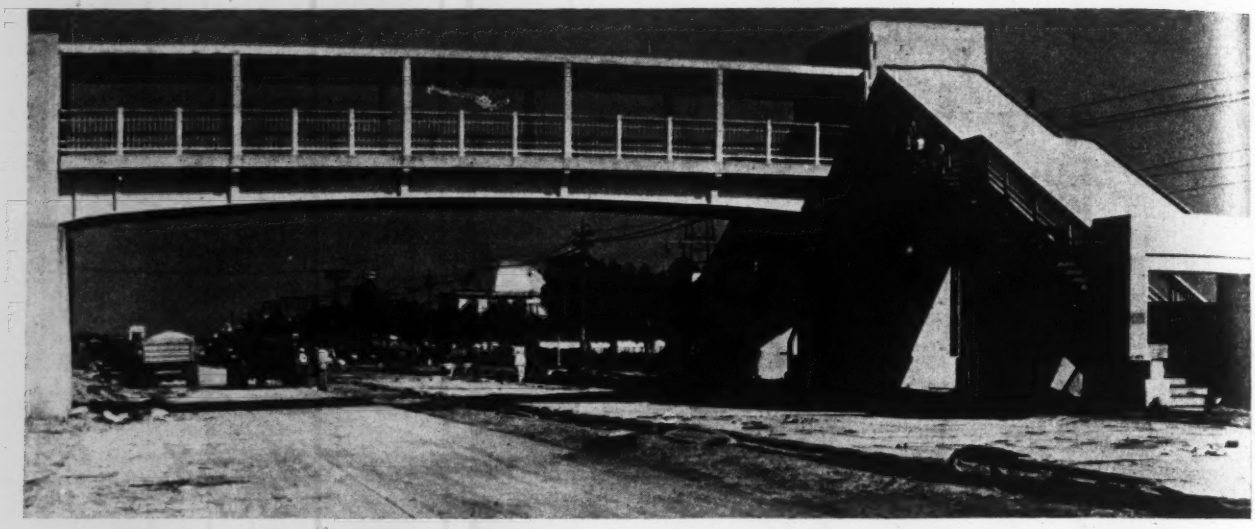


FIG. 1—PEDESTRIAN OVERPASS crosses the five-lane freeway at the Arena in Forest Park on a steel-girder structure with concrete abutments having right and left stairways.

Non-Stop Express Highway Being Built in St. Louis

A three-mile five-lane road without grade-crossings gives access to central business district on new right-of-way

By W. W. Horner
Consulting Engineer, St. Louis, Mo.

THE Oakland Express Highway in St. Louis is probably the first development in the Middle West of a completely segregated freeway. It is being developed to supplement and in part to retrieve the existing major thoroughfare system of the city; it will also provide in part a new road through St. Louis for federal highway 40. Aside from being a fully protected non-stop roadway, other features involved are the use of unencumbered park property for much of the right-of-way, the elimination of several blocks of a near-blighted area, the development of several novel types of under- and overpasses, and clover-leaf crossings at intersections.

The relation of the project to the St. Louis major thoroughfares system and to the central business district is shown on Fig 2. West of the St. Louis limits beyond Skinker Boulevard is situated the most highly developed suburban area of the region. This area embraces the major satellite cities of Clayton (county seat of St. Louis County), Richmond Heights, Brentwood, Webster Groves, Glendale, Kirkwood and the suburban villages of McKnight, Ladue, Deer Creek and Huntleigh. Traffic from a large part of this area, bound for points in St. Louis and particularly for the central business district, uses Clayton Road in St. Louis County and the adjacent streets to the southwest corner of Forest Park. At the present time a large part of it travels diagonally across the park and follows the Lindell-Olive system to the business district (*ENR*, Nov. 15, 1928, p. 731). Another portion of the traffic volume follows Oakland Ave. and the narrow streets east of Kingshighway to reach Market St. or Chouteau Ave.

Market St. from Third to Vandeventer was widened to 100 ft. under the major thoroughfare program of the 1923 bond issue, and the paving of the widened thoroughfare was completed in 1933 (*ENR*, Feb. 22, 1923, p. 332, and June 28, 1934, p. 1126). Vandeventer Ave. had been similarly widened as a part of the same program. It had been proposed at one time that a further widening would be carried out between Vandeventer Ave. and Skinker Road along the lines of Chouteau, Arco and Oakland Aves. Because of the popular reaction against the benefit assessments for the earlier street-widening projects, this last-mentioned widening project had been held in abeyance. The development of the express highway will now make it unnecessary.

On the passage of the National Recovery Act of 1933 a survey was made of an advisable development of state highways within the city to which federal funds might properly be allocated. This particular project was the subject of a report by the writer to the president of the board of public service of the city. After further study it was approved by the city administration and recommended by the mayor to the state highway commission. An investigation by the chief engineer of the state highway department resulted in a favorable report, adoption of the project and an allocation of funds through the Missouri Highway Commission and the Bureau of Public Roads. The first contracts were let in June, 1934, and at the present time all sections of the project are under construction with the exception of the overhead connection across Vandeventer Ave. to the Chouteau Ave. viaduct. It is expected that bids will be received by the state for this section by about the middle of December.

Location and alignment

In policy and general principles of design the project follows closely the recommendations of the committee of the American Society of Civil Engineers for major thoroughfares. Throughout its whole length of 3.2 miles it will be entirely free from traffic crossings, but it will have connection to the principal cross-thoroughfares through the development of right-hand-turn ramps. The site will be fenced and landscaped throughout, and the traffic movement should be entirely free from outside interference of any kind. It is not intended to be a high-speed thoroughfare and will be policed to that end, but its non-stop feature should make it possible to develop a high traffic capacity and an appreciable saving in time of transit. The movement of the expected large volume of traffic through the western half of St. Louis with the complete avoidance of street intersections is expected to afford natural relief to the existing thoroughfares and to have a marked effect in accident reduction.

On the economic side this project is feasible because of the relatively low expense for the land required. The particular route chosen made possible the use of a strip along the lightly developed side of Forest Park. Actually the area taken from the edge of the park for this purpose had been contemplated for street use by the commissioners of the park at the time when it was laid out in 1876. Between Kingshighway and Vandeventer Aves. it was necessary to locate the freeway through private property. Fortunately for the project there existed in this location a strip of low-priced property that had been deteriorating for some time, about to the point of becoming an incipient blighted area. Through this section it was possible to select a location so as to avoid any expensive improvements.

It will be noted from Fig. 3 that in Forest Park the alignment is curved away from the southern edge in order to leave the eastern side of the park at this particular point and to permit continuance through the least expensive property. The curve east of Kingshighway and diagonal crossing at Papin St. were made necessary by the expensive hospital improvement in the block between Papin and Clayton. As shown on Fig 3, a second curve was introduced at Boyle Ave. to avoid expensive industrial improvements and school buildings along the north side of Papin, east of Boyle. However, the result of this shift to the north permitted a location at Vandeventer satisfactorily close to Market St., and at the same time a simple overhead connection to the Chouteau Ave. viaduct.

Throughout the length between Kingshighway and Vandeventer the freeway is developed as a depressed parkway, passing under each of the cross-streets. The complete plan and profile are shown on Fig. 3, which gives in detail the various bridges required to avoid interference with the existing street system. In profile is shown the grade, worked out in a manner to keep the new highway as close to the general surface as possible. This results in an undulating grade about 18 ft. below the street system at crossings, and occasionally not more than 5 ft. below the street system between crossings. In this section the grade is limited to a maximum of about 4 per cent. The vertical curvature and sight distances are in accordance with state highway standards, but the alignment curvature in some cases uses a slightly shorter radius than is customary on open country location.

As originally proposed by the writer, the project provided for a four-lane pavement, but this was later modified, at the suggestion of the Bureau of Public Roads to a five-lane 50-ft. roadway. In the beginning it is understood that the central lane will be used as a permissive passing lane in accordance with ordinary traffic regulation, but in the future it is possible that it will be used for fixed movements, permitting of three lanes eastbound during the morning peak and three lanes westbound during the afternoon and evening hours.

Except in the case of the two diagonal crossovers of local streets, the acquisition of right-of-way involved the taking of a full lot depth, so that the freeway site extends in width from the old street line to the existing alley line. In all cases both the street and the alley are left undisturbed. Fencing, provided along both the street and the alley line, will be worked into the landscape effect. All of the site in excess of the 50-ft. pavement will be developed with turf, shrubbery and trees. On that account it is expected that the location of the freeway through this dis-

FIG. 2—MAJOR THOROUGHFARE SYSTEM of St. Louis and its relation to the central business district and the western highly-developed suburban area.

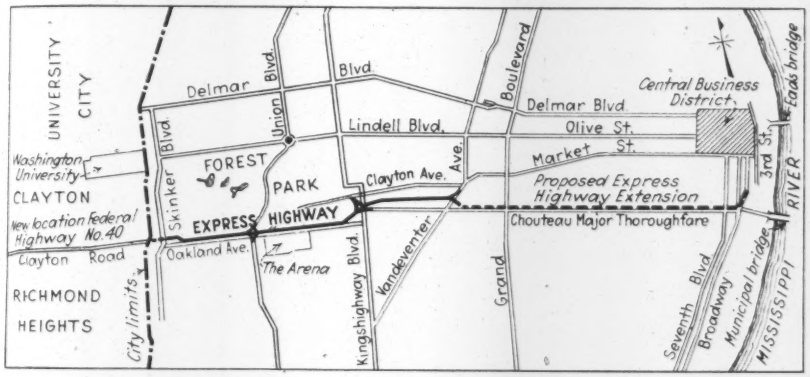
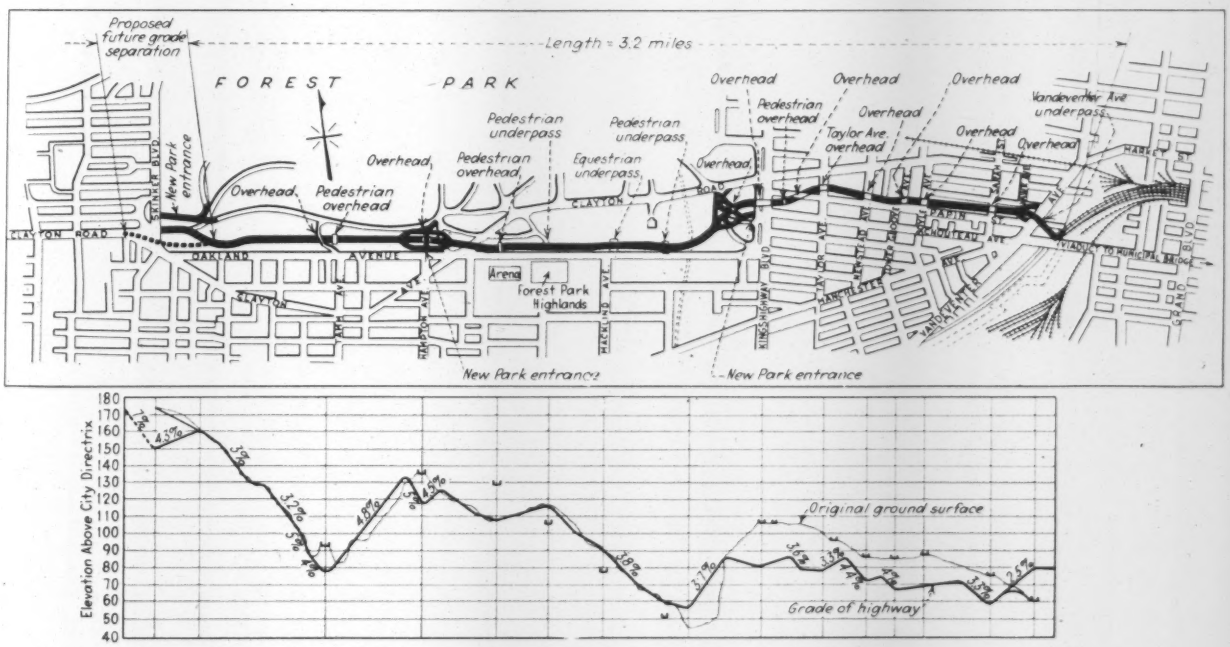


FIG. 3—OAKLAND EXPRESS HIGHWAY traverses south side of Forest Park and parallels Papin Ave. in depressed roadway with all grade-crossings eliminated.



trict will result in an appreciable benefit to the district itself, developing additional light and air for adjacent property, as well as the esthetic improvement resulting from the landscaping.

Under the agreement between the city, the state and the federal government it was necessary for the city to pay the whole cost of acquisition of property. In 1933 the city had funds available from its 1923 bond issue for major thoroughfare development, and because of the urgency of the situation actually began to purchase property without waiting for condemnation proceedings. About 116 separate parcels were involved, and after a quick appraisal an estimate was made that the property could be purchased for $800,000. Under instructions from the city administration the city's right-of-way agent proceeded with the optioning of the property and actually secured about 90 parcels at the prices of the city's original appraisal. Others were purchased at slightly increased costs, and on only one of the 116 pieces was it necessary to apply for a finding of the condemnation commission. The actual cost of acquiring the property is approximately $825,000, an amount only about 30 per cent in excess of the present reduced assessed valuations. In contrast with the older method of condemning property for street-widening, where the actual cost has been from two to three times the assessed valuation, this represents a rather unusual experience. In part the low cost is undoubtedly due to the existing depressed real estate market, but in a large measure it is due to the effectiveness and personality of the city's agents.

Roadway structures

Because the highway is completely segregated for vehicular use it was necessary to develop a large number of crossings in order to maintain access between the Oakland Ave. frontage and the adjacent park area. This resulted in the provision for three overhead vehicular crossings of the highway: one at the Tamm Ave. entrance, a major crossing with full clover-leaf approaches at the Hampton Ave. thoroughfare, and one at the Jefferson Drive immediately west of Kingshighway.

In connection with the Hampton Ave. crossing there is involved a complete new park entrance development, and the realignment of Jefferson Drive together with the ramp connections immediately west of it, are so worked out that a new park entrance results, facing Chouteau Ave. These two improvements will add appreciably to the appearance and convenience of the park in the vicinity of their location.

In addition to the vehicular crossings over the highway a pedestrian overpass is provided near Tamm Ave. on the main park walk leading to the Art Museum. In order to furnish a satisfactory connection between the Arena and the Forest Park Highlands amusement park on the south side of Oakland and the parking facilities inside of Forest Park, two pedestrian crossings are laid out in this vicinity, one an overpass completely covered and the other an underpass.

Another underpass is provided connecting the park police station with the streetcar facilities, and at Macklind Ave. an equestrian underpass has been worked out in order not to interfere with the connection between the riding stables on the south side of Oakland and the bridle paths in Forest Park.

Immediately west of Jefferson Drive in Forest Park the highway crosses a wide valley through which the River des Pères originally ran. This stream was enclosed in a large sewer in 1928 (*ENR*, July 24, July 31, Aug 14 and 21, 1930, pp. 124, 176, 250 and 296 respectively), and advantage of this situation is taken to dispose of the surplus earth from the excavation east of Kingshighway so as to form a large bowl stadium in the valley. The facility for disposing of surplus earth in this manner was an important factor in reducing the cost of the project.

The character of the structures varies considerably throughout the job to meet particular conditions. At Hampton Ave. the long overhead crossing of the clover-leaf involves two 37-ft. and one 81-ft. span of the steel-girder type on concrete bents. The long skew crossing at Papin St. is developed through the use of trusses on the curb line with a concrete deck at about mid-height and cantilevered sidewalks. The Kingshighway over-crossing is a 54-ft. concrete rigid-frame structure with cantilevered abutments, while most of the other over-crossings are rigid-frame structures of steel, with concrete abutments, retaining walls and deck and railings.

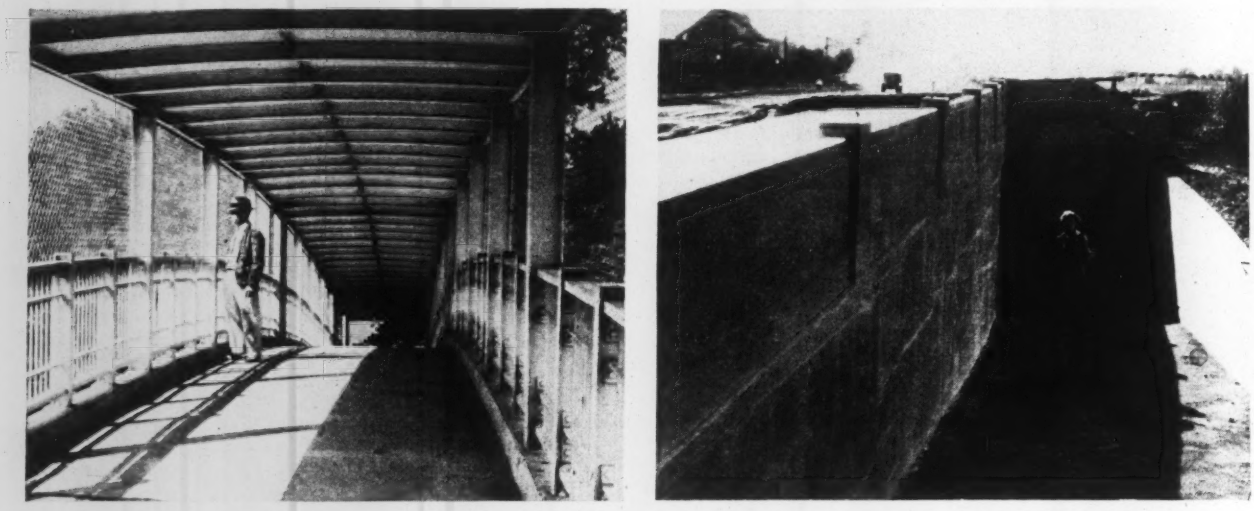


FIG. 4—LEFT: interior of pedestrian overpass heavily screened to prevent children climbing over rails. Right: equestrian underpass to park bridle paths.

The drainage through Forest Park presented no difficult problems, but it required the construction of some long storm-sewer lines to lead the drainage from the highway slab and shoulders to the water courses or sewers in Forest Park. East of Kingshighway the crossings of the Express Highway under the city streets are generally below the level of the city sewer system. At both Taylor and Newstead Aves. important city sewers of the combined type are cut, but in both instances it was possible to avoid siphons by detouring the sewers through the hump in the highway grade between these two streets.

For the drainage of the cut itself east of Kingshighway, it was necessary to construct an independent storm-sewer system beginning at a point near Papin St. and discharging into a city sewer of available capacity about half a block west of Vandeventer Ave. This sewer was constructed below the highway subgrade in advance of paving.

Preparation of the final plans has been carried out by the state highway department with the cooperation of the city's

engineers. The work has been let in five sections at a total contract cost of $1,242,376. There remains, however, the overhead connection of the Chouteau Ave. viaduct on which the estimated cost is $300,000. It is expected accordingly that the total cost of the project, including the acquisition of the property, will be slightly less than $2,400,000. On this basis the project does not exceed the cost per mile of the major thoroughfare widenings carried out in the past, and its traffic capacity will far exceed that of the ordinary 76-ft. roadway which serves also abutting property. As of Oct. 1 the actual construction within Forest Park approached completion, all bridge work having been finished. The 54-ft.-span, 134-ft.-wide bridge at Kingshighway was about 75 per cent completed, and both grading and bridge work were under way between Kingshighway and Vandeventer. It seems possible that the whole project can be opened to traffic, including the Chouteau Ave. connection, during the summer of 1936.

At the same time the state highway department is proceeding with the work on its new short connection of highway 40 into St. Louis, which involves an express type of construction in parts of St. Louis County and a new bridge across the Missouri River. For the present this state highway in St. Louis County will utilize Clayton Road for several miles west of the city limits.

For the three miles of work already under contract the expenditures may be divided into five principal parts: grading, including removal of pavements, $280,000; pavement, $305,000; drainage, $141,000; fencing, $34,000; structures, $478,000.

As the traffic on practically all of the newly widened major thoroughfares leading west from the central business district has now reached a volume that approaches capacity during the evening rush hour, it is expected that the development of the new traffic facility west of Vandeventer will congest both Market St. and Chouteau Ave. east of Vandeventer to an unbearable degree, and it is anticipated that sooner or later it will be necessary to continue the segregated freeway eastwardly from Vandeventer Ave. to about Fourth St. A preliminary plan for such an extension has been prepared in the form of a four-lane elevated structure in Chouteau Ave. with connection both to the Municipal Bridge and to the eastern side of the business district. This plan has been given serious consideration by the state highway department. The estimated cost is $4,000,000, and it seems probable that sufficient funds will be available in the St. Louis district to permit the work to be undertaken at the present time.

The work is being carried out by the Missouri State Highway Commission, of which Scott Wilson is chairman and T. H. Cutler chief engineer. The detail plans were prepared in the St. Louis district office of the highway commission under Paul Daniells, district engineer. The designs of the Kingshighway bridge and of the Vandeventer-Chouteau connection have been prepared by Sverdrup & Parcel, consulting engineers, for the state highway commission, and all other structures were designed in the bridge division of the commission under N. R. Sack, bridge engineer. The project has been constructed by a cooperative arrangement with the city of St. Louis under Bernard F. Dickman, mayor, Baxter L. Brown, president of the Board of Public Service, L. A. Pettus, division engineer of sewers and paving, and William C. Doerr, right-of-way agent.

The project was outlined by the writer in July, 1933, while engineer for the city of St. Louis, and he has continued on the design as advisory consulting engineer to the state highway commission, also preparing detailed plans for the revision of the city sewer system and the drainage of the cut east of Kingshighway.

Montreal to Build New Intakes for St. Lawrence River Water

Six lines of 7-ft.-diameter concrete pipe to extend 2,200 ft. out into the river—Present intakes are of inadequate capacity and tap polluted silt-bearing waters near the shore

CONSTRUCTION of a new water intake extending 2,200 ft. out into the St. Lawrence River is proposed by the city of Montreal, and bids have been taken but no contract let because of the limited funds available. The decision to call for bids on the construction of six lines of 7-ft.-diameter concrete pipe to supplement the existing pipe intakes came following a long discussion of whether the expense involved, estimated at $1,200,000, could be avoided by improving the treatment given to the water in the water-treatment plant or whether the capacity of existing intakes could be increased sufficiently to meet current demands.

All water used by the city is drawn from the St. Lawrence River at Ville LaSalle, a small municipality on the island of Montreal, at a point about 8 miles upstream from the Montreal harbor and just at the head of the Lachine Rapids, the first rapids on the river above tidewater.

The intake arrangement consists of two submerged pipe lines that come together under a small intake gatehouse at the head of an aqueduct, an open channel extending from Ville LaSalle to the St. Joseph Street filtration and pumping plant located near the boundary of the cities of Montreal and Verdun. The latter is an important suburban municipality.

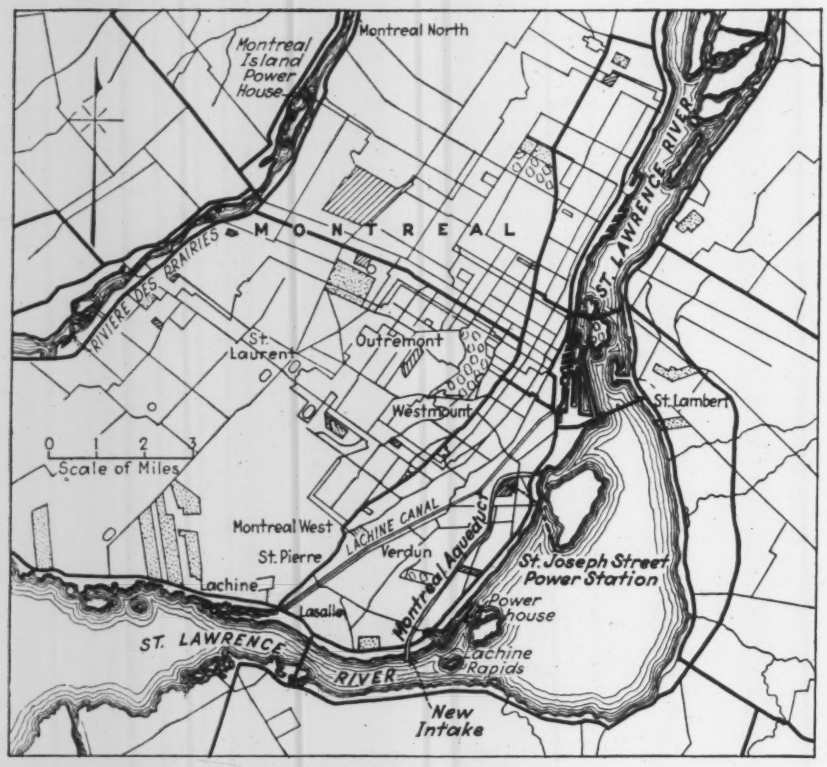
In addition to the two river intake pipes, 6-ft. in diameter and 1,200 ft. long, extending well out into the main stream, there are two subsidiary pipes only 600 ft. long which, owing to their location, collect water from close to the shore line. The water so obtained is polluted, this pollution having increased with the increase of population along the shore of the river above the intake, which has been quite marked within recent years. In addition, the water near the shore consists mainly of the flow of the Ottawa River, which joins the St. Lawrence several miles above the intake, but which still maintains its individual character at the intake point, the most marked feature of which is its heavy silt content as distinct from the clear water of the St. Lawrence River. The result of this condition was very marked during the spring of 1935, when the water supplied to the city contained so much silt as to be characterized as "yellow water."

The capacity of the two pipes of the main river intake is only 80,000,000 gal. per day, with a loss of head of about 5 ft., as compared with the pumping installation of the city, which is laid out for an ultimate capacity of 300,000,000 gal. per day.

All water from the aqueduct is passed through sand filters at the St. Joseph Street plant, is chlorinated and pumped to the several service reservoirs serving the city, generally located on the slopes of Mt. Royal. Other pumping stations are associated with the service reservoir.

Several proposals considered

The unsatisfactory condition of the water-supply system has repeatedly been stressed in the reports of the chief engineer of the Montreal Water Board. As far back as 1921, it was recommended that the intakes be increased in capacity to correspond to the rest of the system, and that they be extended far-



THE PROPOSED NEW INTAKE for the water supply of Montreal is to extend a quarter-mile out into the St. Lawrence River above the Lachine Rapids.

ther out from the shore. This was before the city of Montreal purchased the Montreal Water & Power Co. and, in 1926, set up the Montreal Water Board to serve the city and adjoining municipalities.

In 1934 the city authorities were advised of the serious pollution of the water then being used, and estimates were prepared which showed that if the whole supply to the city were treated adequately, the annual cost of coagulants alone would equal the interest charges on an investment of $3,600,000, which was greatly in excess of the total cost of the proposed new intake. Finally, early in 1935, when the annual bill for the city of Montreal was before the provincial legislature of Quebec, authority was obtained to seek power to raise a loan for the construction of a new intake.

Discussion, however, still continued. The idea of employing settling basins was advocated, but the cost of these was shown to be at least $2,000,000. Eventually, in order to come to a final decision, the city council appointed a board of five engineers to review the plans and estimates of the chief engineer of the water board and to report specifically on the following questions: (1) whether the condition of the intake at present in use is really serious; (2) whether the intake could be repaired effectively and economically, or whether a new intake is necessary; (3) whether, if repairs could be undertaken, the existing intake could be extended at a cost of $350,000, as had been suggested to the council; and (4) whether, if a new intake has to be constructed, four pipes instead of six, as suggested by the chief engineer of the water board, would not be adequate; also, whether the intake could be constructed more cheaply by tunneling out from the shore, instead of laying pipes on the riverbed.

The report of the board of engineers was submitted to the city council on Sept. 27. Its contents have not been made public, but it is known that the plans and proposals of the chief engineer of the water board were completely endorsed. On Oct. 3 the council called for bids for the construction of an entirely new intake structure. These bids were opened on Oct. 30, at which time it was found that a combination of the lowest bids for an intake of six pipes was in excess of $2,000,000 as against the authorized expenditure of $1,200,000. At the time of the opening of the bids two of the bidders proposed to do the job by tunneling at a cost within the engineer's estimate.

The executive committee of the city of Montreal more recently referred the matter back to the water board and instructed that new tenders be called for, covering both the supply and installation of the concrete pipes suggested in the official plans of the board's engineers, considered as one contract. The proposal to construct a tunnel intake was not mentioned by the executive committee in its communication to the board.

New intake structure

Under the plan according to which bids were called, construction of the new intake is to be handled as two separate contracts, one for the manufacture and supply of the 7-ft.-diameter reinforced concrete pipe and the other for the installation of the pipe and associated work. Briefly, the new intake as previously noted, will consist of six lines of pipe laid in a specially prepared trench on the bed of the river and extending out at right angles from the shore for a distance of 2,200 ft. This will bring the intake head into the clear waters of the St. Lawrence River as distinct from the shore waters coming from the Ottawa River. The six pipes will lead directly into the bed of the aqueduct, discharging into the canal a short distance below the present upper end of the channel. (The aqueduct is a wide channel with head gates shutting it off from the river, built with the idea of using it to divert water from the river for both water supply and power development.)

Flow through the intake pipes will be controlled from a gatehouse located some distance offshore but behind the breakwater protecting the aqueduct headworks. This structure will be of reinforced concrete, 92x45 ft., founded on rock 42 ft. below the floor level and 22 ft. below water level. It will be equipped with twelve 6x6-ft. wedge sluice gates, motor-operated.

At the outer end of each pipe there is to be an intake manifold of six openings built at right angles to the line of pipes and lying in the direction of flow of the river. These right-angle branches containing the manifold will be spaced at 72-ft. centers.

The present plans call for the pipes to be laid in still-water chambers formed by cofferdams of steel sheet-piling, the cofferdams to be extended out from the shore in successive steps.

During construction, the existing intake will continue to be used, the necessary cofferdams within the canal being so located as to leave the intake flumes free of interference. The shore end of the new work will be tied in with existing embankments and with the stone-filled timber-crib breakwater protecting the present works. The offshore work, however, will be conducted in relatively swift water until the stilling chambers are formed.

The Montreal Water Board is a public body composed of members of the city council with Alderman Georges Caron as chairman. J. E. Blanchard is director of works for the city of Montreal, and C. J. Des Baillets is chief engineer of the water board. Mr. Des Baillets prepared the plans for the new work and will supervise its execution.

Construction Costs Analyzed on Large Apartment Building

Breakdown of all costs on Knickerbocker Village housing project in New York shows direct labor gets 32½ per cent of building dollar, while 42.4 per cent goes for materials and 25.1 per cent is for overhead and miscellaneous items

LOW-COST HOUSING DEVELOPMENTS have become a major item in the present administration's social and employment relief programs. The government, through the Bureau of Labor Statistics, selected the Knickbocker Village housing development in New York City, financed by the Reconstruction Finance Corporation, as a test project in determining the the cost of housing, thereby analyzing their value as social improvements, and in determining the work-creating potentialities of building construction of this type. In the analysis the building costs were broken down into nineteen construction and three overhead items, and the direct employment provided by each construction item was compiled. The results of the analysis were published in the September, 1935, issue of the *Monthly Labor Review* of the Bureau of Labor Statistics. All information included in the following was abstracted from that publication.—EDITOR.

CONSTRUCTION of a modern apartment house in New York City in 1934 cost 46.8c. per cu.ft. and $4.86 per sq.ft. of floor space, according to a breakdown of costs on the Knickerbocker Village two-unit housing development made by the Division of Construction and Public Employment of the Department of Labor. The total construction cost was $6,216,899, of which $2,019,838 (32.49 per cent) went for direct labor at the site, $2,636,179 (42.4 per cent) went for materials and $1,560,882 (25.11 per cent) was spent for overhead and miscellaneous items. Employment at the site totaled 1,759,600 man-hours.

Knickerbocker Village was built by the Fred F. French Co., with the aid of a loan of $8,022,000 from the RFC. The project was started in 1933, and one unit was ready for occupancy Sept. 25, 1934, and the other unit in December of the same year. The total cost of the development, including land and buildings, exceeded $9,500,000. Both buildings are twelve-story red brick structures, and together are capable of housing in 1,593 dwelling units a population of approximately 6,000. The apartments were designed primarily for white-collar workers. Monthly rentals average $12.50 per room. A number of thoroughly modern 2½-room apartments, however, rent for $22.50 per month. The highest rent is $87.50 for a five-room penthouse apartment.

Cost analysis

In making the cost analysis it was found that wide differences in the accounting practice of the 240 firms that had a part in the construction of the building required close adherence to rigid and sometimes arbitrary classifications. Labor costs include only labor performed at the site of construction. Material costs cover the cost of the materials laid down at the site, including freight and drayage charges and all shop labor required in fabrication. Overhead charges included workmen's compensation, amounting to approximately 10 per cent of labor cost, and other costs, such as maintaining and hiring concrete mixers, hoists, steam shovels and other equipment.

The costs are detailed in the accompanying table. The 22 classes of work and services represent the complete cost of construction except for that of demolishing the tenements which formerly occupied the site. In the original article the cost per cubic foot was given as 46.8c. and the cost per square foot as $2.43. This indicated a ceiling height of approximately 5 ft., and an inquiry to the Bureau of Labor Statistics brought forth the astounding reply that the square-foot costs were based on the combined area of floors and ceilings. The true cost per square foot of floor space is therefore $4.86, and this figure has been used in our table.

In analyzing the various cost items it will be noticed that masonry, accounting for 15.3 per cent of the total cost, was the largest item in both labor and material cost and provided over 400,000 man-hours of employment. Cement and concrete work was next, costing 11.6 per cent of the total and providing 242,500 man-hours of employment.

Where the building dollar goes

The construction cost of the building totaled $6,216,899, and with 1,593 apartments this represented a cost of $3,903 per unit. Computed on the basis of 5,377 rooms, the average cost for each room amounted to $1,165. Each room provided 330 man-hours of employment.

On this project the largest part of the building dollar, 15.31c., was spent for masonry work, and the smallest part, 0.25c., was expended in glazing. Five items, including masonry, accounted for over half the building dollar; these were cement and concrete at 11.64c., structural steel at 9.91c., plumbing at 9.31c., and carpentry at 6.31c. In addition to glazing, two groups, weatherproofing and roofing and sheet-metal work, each accounted for less than 1c. out of each dollar spent. Other items accounting for a small percentage of the dollar were tile and terrazzo at 1.45c., engineering and supervision at 1.75c., architect's fee at 1.77c., and equipment at 2.30c. In the lathing and plastering group 5.94c. was expended, in heating and ventilating 4.53c., and in manufac-

COST DISTRIBUTION—KNICKERBOCKER VILLAGE HOUSING PROJECT, NEW YORK

Item Classification	Total Cost		Direct Labor Cost		Materials		Overhead & Misc.		Unit Cost		Thous. Man-Hr. at Site
	Amount	Per Cent of Total	Amount	Per Cent of Class	Amount	Per Cent of Class	Amount	Per Cent of Class	Per Cu. Ft.	Per Sq. Ft.	
All classes of work	**$6,216,899**	**100.0**	**$2,019,838**	**32.49**	**$2,636,179**	**42.40**	**$1,560,882**	**25.11**	**$0.468**	**$4.86**	**1,759.6**
Excav. and grading	204.908	3.2	89.677	43.76	8.173	3.99	107.058	52.25	.015	.156	101.1
Cement and concrete	723.363	11.6	273.372	37.79	345.417	47.75	104.574	14.46	.055	.570	242.5
Masonry	951.830	15.3	466.281	48.99	423.376	44.48	62.173	6.53	.072	.748	408.0
Carpentry	392.515	6.3	119.161	30.36	226.553	57.72	46.801	11.92	.030	.310	94.9
Plumbing	578.929	9.3	178.626	30.85	344.375	59.48	55.928	9.67	.044	.456	137.6
Heating and ventilating	281.228	4.5	82.721	29.41	176.598	62.80	21.909	7.79	.021	.218	67.2
Lathing and plastering	369.157	5.9	235.548	63.81	93.709	25.38	39.900	10.81	.028	.290	187.0
Roofing and sheet metal	29.091	.5	13.070	44.93	12.938	44.47	3.083	10.60	.002	.028	10.1
Structural steel	616.209	9.9	86.947	14.11	376.167	61.05	153.095	24.84	.046	.476	53.6
Electrical work and fixtures	230.272	3.8	92.441	40.14	110.630	48.04	27.211	11.82	.017	.176	70.6
Painting and decorating	150.120	2.4	83.283	55.48	36.534	24.34	30.303	20.18	.011	.114	65.7
Elevators	157.120	2.5	25.552	16.26	130.866	83.29	702	.45	.012	.124	24.4
Tile, terrazzo, bathroom fixtures	90.178	1.4	40.194	44.57	39.759	44.09	10.225	11.34	.007	.072	33.2
Weatherproofing	55.918	.9	29.410	52.59	23.862	42.67	2.646	4.74	.004	.042	25.1
Mfd. metal products	232.070	3.8	31.830	13.72	154.357	66.51	45.883	19.77	.017	.176	26.0
Glazing	15.382	.2	4.472	29.07	4.982	32.39	5.928	38.54	.001	.010	4.4
Equipment	142.782	2.3	2.312	1.62	119.037	83.37	21.433	15.01	.011	.114	1.8
Engineering and supervision	108.640	1.7	92.113	84.79	1.060	.98	15.467	14.23	.008	.084	61.6
Miscellaneous	173.003	2.8	72.828	42.10	7.786	4.50	92.389	53.40	.013	.134	144.8
Builder's fee	340.098	5.5					340.098	100.00	.026	.270	
Architect's fee	109.901	1.8					109.901	100.00	.008	.084	
Financial and other charges	264.185	4.4					264.185	100.00	.020	.208	

tured metal products 3.73c. Out of each dollar the builder's fee accounted for 5.47c., and financial and other charges took 4.25c. The remaining group, which included electric and electrical fixtures, excavation and grading, miscellaneous, elevator, and painting and decorating, accounted for 14.72c. of the dollar.

Construction details

The building is of steel-frame construction with exterior walls of tapestry brick trimmed with architectural terra cotta. Inside partitions are of gypsum and terra cotta blocks. Of the total labor costs for masonry, amounting to $466,281, exterior walls accounted for $285,077, and $172,996 in wages was charged for inside partition construction and $8,208 for scaffolding.

Heating and ventilating together totaled $281,228. Heating costs, which included central heating-plant equipment, were $255,528; ventilation work was confined to sheet-metal ducts in the basement and amounted to only $14,000. A large steel smokestack cost $3,700, including $900 for rigging expenses. An unusual charge of $8,000 for repairs necessitated by breaks in expansion joints is contained in the total. Wire lath was used throughout, no furring or lathing being required on exterior walls because of the use of weatherproofing material.

The roof was constructed of slag, and gutters and rain conductors, similar sheet-metal work and a small amount of roofing tile were included in the roofing contract.

Painting and decorating contracts covered the painting of all interior walls as a substitute for wall paper. Window shades and venetian blinds were included in the decorating cost. Self-operating passenger elevators were installed, and the building was equipped with ash-hoist elevators. Temporary elevators, used for transportation of materials during construction, were included in the elevator cost.

Equipment items include electric refrigerators, gas ranges, laundry dryers and miscellaneous fixtures.

Primary Treatment Plant Includes Magnetite Filter

Mechanical straining of settled sewage at Goshen, Ind. will give additional clarification at reasonable cost in plant now under construction—Intercepting sewer system designed to improve heavily polluted streams

By Walter S. Bemis
Consulting Engineer, Chicago, Ill.

MECHANICAL filtration of settled sewage has been adopted in the primary treatment of sewage for the plant at Goshen, Ind,, where partial treatment was considered sufficient to clean up the first major point of pollution on the watershed of the Elkhart and St. Joseph Rivers until such time as other towns below had complied with the state board of health orders. The use of this type of filter was considered after observation of an experimental unit at the North Side plant of the Sanitary District of Chicago, which operated on an activated-sludge effluent.

An objection to filtering fresh sewage seemed to be the possibility of developing septic conditions in the filter sand unless extensive pre-aeration were used. To satisfy the writer's apprehension, a series of tests was run at an experimental plant in Coney Island, N. Y., which indicated that nothing was to be feared from this source. The primary reason for installing a mechanical filter was that by its use there will be an increase of about 25 per cent in the removal of suspended solids at reasonable cost over the ordinary primary treatment methods. It was also recognized that the filter would be adaptable to greater usefulness in the future when a higher degree of treatment became necessary, involving chemical treatment, trickling filters or activated sludge.

The treatment plant is comprised of a pumping station, screenings shredder, mechanical grit remover, settling tank provided with a magnetic filter and rotary sludge scraper, a digester and open sludge beds.

Downstream from Goshen the river system receives gross pollution from Elkhart, Mishawaka and South Bend in Indiana and then crosses over into Michigan to be further polluted by Niles and other smaller communities until it flows into Lake Michigan at St. Joseph, Mich. None of these cities treats its sewage. Therefore, although these waters flow through scenic and historic surroundings, their pollution has been permitted to continue as a menace to health, thus restricting their value from the standpoint of recreation and esthetics. At points of greatest concentration of sewage the condition of the river waters has been a source of nu[illegible]ce and has been the subject of comp[illegible]ts. The municipalities downstream [illegible]ve been reluctant to proceed with any [illegible]n of sewage disposal until their neig[illegible]rs upstream have cleared up their poll[illegible]on. When the state of Michigan threa[illegible]ed to take legal action to stop the poll[illegible]on of the waters of the Elkhart and St. Joseph Rivers before they flow into Michigan, the State Board of Hea[illegible]h of Indiana requested Goshen to act promptly toward eliminating the discharge of untreated sewage into the Elkhart River.

The monthly average flow of the Elkhart River at Goshen varies between [illegible]0 and 2,300 sec.-ft. The normal monthly summer flow averages 200 sec.-ft. Upstream are several hydro-electric plants that cause some diurnal fluctuations during low flows.

The projected system

The sewerage system of Goshen is of the combined type consisting of mains and laterals laid about 50 years ago, to which additions were made from time to time. Discharge is at four main points into the Elkhart River and at five lesser points into Rock Run Creek, a tributary flowing into the Elkhart River a short distance downstream from Goshen. In addition to the above, there are a number of private outlets from residences and industries located along the banks of the Elkhart River and Rock Run Creek

In the newly proposed scheme, which is now under construction and is expected to be in operation early in 1936, it was found necessary to build an intercepting system and a treatment plant. Some laterals were included in the program to take care of unsewered territory contiguous to the intercepters. The intercepting sewer system is essentially high level, with the sewage flowing by gravity to the treatment plant, where it is lifted 14 ft. above the sewer invert to take care of the head losses in the plant. The outfall sewer carrying the effluent from the plant to the river is a continuation of the intercepter and can be used to bypass the entire plant in an emergency.

The interceptor system consists of two main lines, one through the heart of the town paralleling an old main sewer and the other following the banks of Rock Run Creek, to collect the sewage now being discharged into that grossly polluted stream. Three small pumping stations are being built to lift the sewage from lower areas into the trunk sewers. The two main sewers join and continue as a single line for 4,000 ft. to the plant as a 42-in. precast concrete line. The total length of interceptors, laterals and outfall sewers under construction is 37,400 ft., ranging in size from 8 to 60 in. in diameter.

The plant site, located northwest of the city just outside the city limits, was chosen because of its availability, isola-

tion and hydraulic advantages. It is on the east bank of the Elkhart River and has an area of 7.3 acres, which is sufficient for all future needs of the plant, including the instalaltion of full secondary treatment if, as and when required.

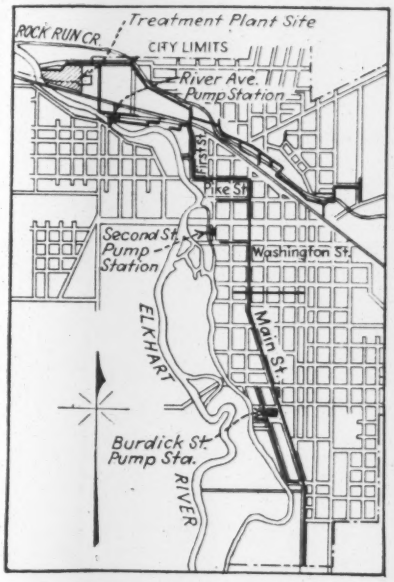
Goshen has a population of 11,000 people, of which 10,000 are provided with sewerage facilities. The design was based on an ultimate tributary population of 16,000 expected in 1955.

Selection of treatment

Although primary treatment is considered ample to meet the present available dilution in the Elkhart River, provision has been made in the plant design and construction so that complete treatment can be provided with a minimum of new construction. As an additional safeguard to protect the esthetic, recreational and health facilities of the river, particularly in the immediate vicinity of Goshen, it was deemed advisable to permit a minimum of suspended solids to reach the river. The magnetite filter placed on the sedimentation tank is expected to increase the suspended solids removal by 25 per cent over that obtained by sedimentation alone. The placing of the filter unit on the sedimentation tank will minimize the possibility of sludge accumulations in the river, will provide a high-grade treatment at reasonable cost and will postpone the need for secondary treatment devices. The design and construction of the plant contemplate the future addition of chemical treatment, trickling filters or activated sludge.

Design factors

The intercepting sewer to the plant has a capacity of 20 m.g.d. The pumping station is designed for a present capacity of 4.45 m.g.d. and a future capacity, after adding one pumping unit, of 7.32 m.g.d. This is more than four times the ultimate dry-weather flow (175 m.g.d.) expected in 1955. An annual average flow of 2.25 m.g.d., expected by the year 1955, is equivalent to a per-capita flow allowance of 140 gal. daily. The final 4,000 ft. of 42-in. interceptor to the plant has been so designed that it can be ued for storage of ordinary stormflows, thus flattening out the maximum flows reaching the plant.



HIGH-LEVEL interceptor-sewer system with three booster pumping stations eliminates former discharge outlets into Elkhart River and Rock Run Creek.

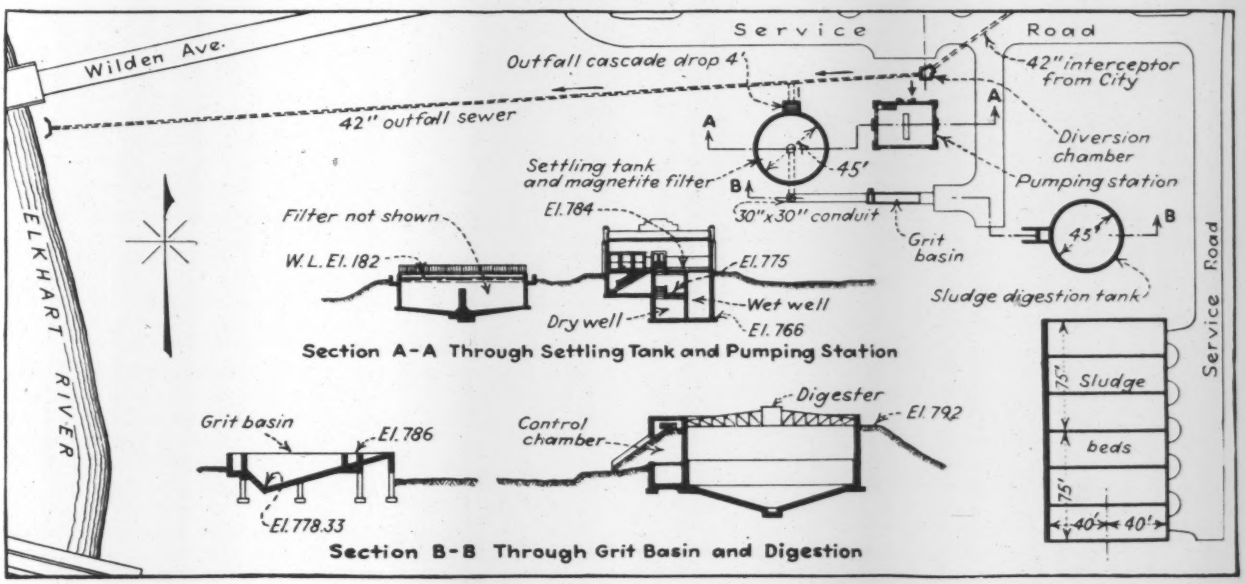
Water levels in the Elkhart River control the plant elevation. High water has been recorded at El. 780, and the average river levels during dry-weather flows approximate El. 772 ft. The invert of the incoming 42-in. interceptor at the plant is at El. 771. In order to be above high-water levels in the river, the flow line in the grit chamber has been placed at El. 783.5, and a drop of 0.5 ft. has been provided from the grit chamber to the primary tanks, thus allowing the plant to operate at all stages of the river. The average ground surface in the vicinity of the plant is being kept at a minimum elevation of 780 ft. for the same reason. The sludge beds have been set at El. 778.5, and the flow line of the digester at El. 794.

Equipment and treatment devices

A comminutor or shredder (7-m.g.d. capacity), with 1-in. clear openings and automatic control, has been installed for the purpose of chopping all bulky matter into fine pieces and returning the fines thus produced to the sewage flow in the wet well. In parallel with this equipment for relief or supplementary operation is a hand-cleaned bar screen with openings of $1\frac{3}{4}$ in. between the bars.

The grit remover consists of a single-unit mechanism with a maximum capacity of 7.2 m.g.d. The channel in which the grit remover works is 34 ft. long and has a variable side water depth of 1.85 ft. A novel feature introduced in this design consists of a proportional weir to maintain a constant velocity of sewage going through the chamber for all volumes of flow, thus eliminating the expense of constructing the usual rectangular gravity grit basin with associated mechanism.

The primary settling tank is 45 ft. in diameter and has a 10-ft. side water depth. It is equipped with rotary sludge scrapers and with an automatic magnetite filter 6 ft. wide. This tank has an average detention period of $1\frac{1}{2}$ hr., which becomes 2 hr. during dry-weather



GOSHEN'S partial sewage-treatment works comprises a pumping station and screen house, grit chamber, settling tank with filter attached, separate sludge digester and sand-drying beds.

GOSHEN SEWAGE-TREATMENT PLANT under construction. In foreground is narrow grit chamber with proportional weir for constant velocity, in background is primary settling tank with mechanical filter, and at right is the pump house.

conditions and 0.75 hr. at the maximum rate of flow. The settling area of the tank is 1,589 sq.ft., and the filter bed has an area of 735 sq.ft.

The filter is designed to operate under average flow conditions of 2.1 gal. per sq.ft. of filter area per minute, under maximum pumping conditions at a rate of 4.2 gal. and during dry-weather flows at a rate of 1.65 gal. per sq.ft. of filter area per minute. The filter consists of a 3-in. depth of magnetite sand held in a trough with concrete sides. The bottom of this trough is composed of a phosphor-bronze screen of woven wire with 0.25-in. openings supported on a steel grating, similar in design to that used at the plants at Dearborn, Mich., and Perth Amboy, N. J. The sand (magnetite) is sized so that 100 per cent passes a 10-mesh screen and 90 per cent is retained on a standard 23-mesh screen.

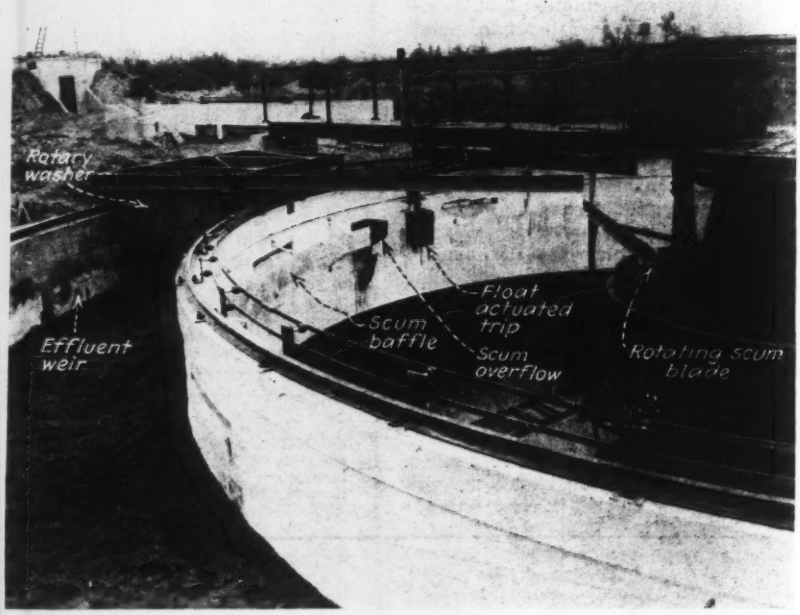
Sewage enters the settling tank through a center feed arrangement, and the settled liquor flows upward through the filter, which traps the non-settling suspended solids. Clogging of the bed causes an increase in head in the middle of the tank, which in turn causes a float arrangement to start the cleaner mechanism automatically. The cleaner consists of a solenoid, washwater compartment and washwater pump, all mounted on a car that travels on rails over the filter. The solenoid, intermittently picks up a section of magnetite sand about 4 in. wide across the filter bed and allows the water from beneath the screen to wash this magnetite and flow over into the pump compartment, from which the dirty water is returned to the center of the clarifier. The cleaner stops when the bed becomes clean and the loss of head through the filter drops. The normal loss of head through the filter bed is about 1 in. and may reach a maximum of 3 in. when clogged. It is found that non-settling suspended solids, after being trapped in the filter bed and returned to the tank influent, settle readily.

ent population. The beds can be easily extended when the need arises.

Pumping stations

At present the plant pumping station will have three vertical, electrically-driven centrifugal pumps of the following capacities: 650 g.p.m at 18-ft. head, 900 g.p.m. at 20-ft. head, and 1,600 g.p.m. at 22-ft head. Space has been provided for installing a future unit with a capacity of 2,100 g.p.m.

Three pumping stations are being constructed to elevate the sewage from low areas to the interceptors, as it was found to be more economical to handle the sewage this way than to lower the interceptors. The River Ave. pumping stations has two pumps of 220- and 400-g.p.m. capacity, respectively, with an average lift of 19 ft. The Second St.



THE UNDERFED FILTER UNIT is supported on a shelf in the primary settling tanks, and operation of the solenoid is controlled by a float mechanism actuated by loss-of-head changes. Supported from walkway by chains is a scum guide which, when lowered, will divert scum to a square overflow at end of scum baffle.

The sludge-pumping equipment consists of one electrically-driven pump with a capacity up to 60 g.p.m. at a head of 40 ft., and for auxiliary service one portable pump of the same capacity driven by a gasoline engine.

Sludge from the settling tank will be digested in a 45-ft. floating-cover type of heated digester arranged for gas collection. This unit has a capacity of 32,000 cu.ft., or 2 cu.ft. per capita of design population. Gas is used to heat the plant building and the contents of the digester.

Six 25x80-ft. open sludge beds provide 12,000 sq.ft. of drying area, which amounts to 1.2 sq.ft. per capita of present population.

station has two pumps with capacities of 150 and 250 g.p.m. respectively, with lifts of 21 ft. The Burdick St. station has two units, each having a capacity of 150 g.p.m. with a lift of 18 ft.

Construction methods

All of the interceptors were built in open cut except when crossing under railroad tracks. The main line of the New York Central R.R. Co. was crossed by driving 54- and 48-in.-diameter corrugated metal pipe under the two main line and three secondary tracks. A 30-in.-diameter concrete sewer pipe was then placed inside the metal pipe with the invert approximately 20 ft. below the

rail. The void between the shields and the sewer pipe was filled with a concrete saddle and sand except under the two center main-line tracks, where monolithic concrete was used.

Precast concrete pipe was used for all intreceptors over 24 in. in diameter, and vitrified pipe was used for the rest. The concrete pipe was fabricated on the job by the Lewistown Pipe Co., of Fort Wayne, Ind., with labor obtained from the local relief lists.

Particular attention was given to the architectural features of the plant building. The entire exterior is faced with Indiana limestone, and ornamental cast-aluminum doors are hung in the front entrance. All of the other window and door frames are of aluminum construction. The interior main floor is of terrazzo construction; the lower part of the walls is covered with a wainscoating of Tennessee marble, and the upper parts of the walls and the ceiling are covered with California stucco.

Contracts

On Nov. 2, 1934, bids were received and contracts later awarded for the construction of the interceptors and for furnishing f.o.b. Goshen the equipment controlling the principal design features of the disposal plant. The equipment thus contracted for consisted of the screening and shredding units, sludge scrapers for the primary tank, sludge-digestion tank cover and the sludge and sewage pumps.

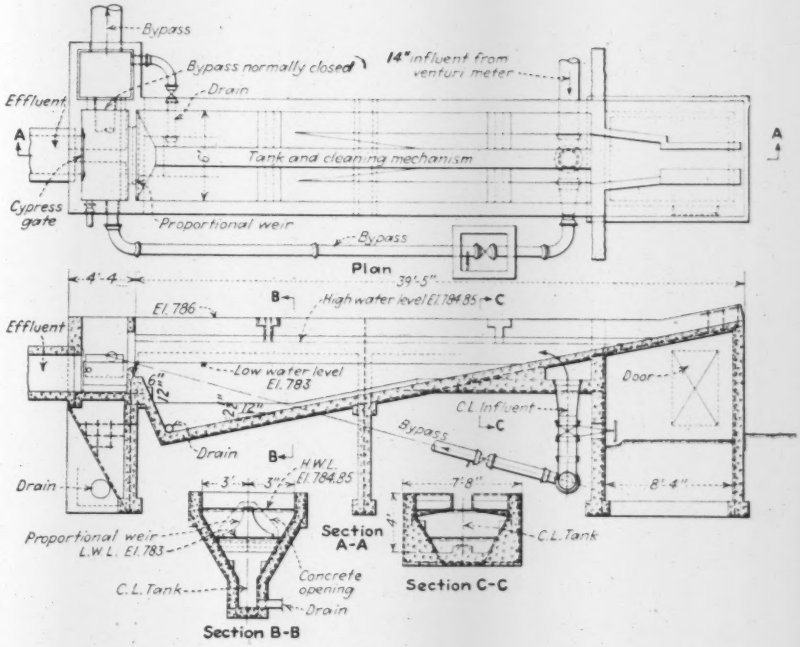
Work on the interceptors, outfalls and appurtenances was let by contract to John Dehner, Inc., of Fort Wayne, Ind. Contracts, also were made for the following equipment: lakeside comminutor, Dorr detritor, primary sedimentation tank equipped with Dorr scrapers and Filtration Equipment Corp. magnetite filter, and a Pacific Flush Tank Co. floating cover for the digester.

The treatment plant was designed around the equipment thus purchased. A separate contract was let to Fluor Brothers and Smith of Oshkosh, Wis., for the construction of the plant and the pumping stations, including the installation of the equipment. In this contract was also included the purchasing and installation of the magnetite filter.

This project is being financed through a PWA loan of $350,000 (bearing 4 per cent interest) and a grant of $105,000. It is being completed well within the $455,000, although the original estimate did not include the filter and a large number of house and street connections necessitated by a paving program started by the Indiana Highway Commission after the letting of the sewer contract.

Roughly, the contract costs may be divided as follows: $270,000 for sewers, $15,000 for three outlying pumping stations, $105,000 for the primary treatment plant, including the main pumping station. These figures include all landscaping, grading, walks, drives, furniture and fixtures but are exclusive of land, engineering, legal and miscellaneous overhead expenses. Sufficient money is still available to provide prechlorination, if desired.

The writer, acting as consulting engineer, prepared the design and supervised the construction, with A. A. Brensley in immediate charge as resident engineer on the job.



A NOVEL FEATURE of the grit-chamber design is the proportional weir to maintain a constant velocity for all volumes of flow. A mechanical grit remover will be installed at a later date.

New Organization Established to Correlate Hydraulic Research

By Herbert N. Eaton
National Hydraulic Laboratory, Washington, D. C.

THE International Association for Research on Hydraulic Structures was founded at Brussels in September, 1935, on the occasion of the meetings of the Permanent International Association of Navigation Congresses. The purpose of the new association is to promote international cooperation in hydraulic research and to facilitate the interchange of the results obtained. The interest of the association will be mainly in the field of hydraulic structures, particularly in connection with model tests, and the verification of the results thus obtained by tests on the full-sized structure. Hydraulic machinery and ship-model testing are not included in the field of the association's interest.

The scope of the activities of the association cannot be predicted at the present time, as this will depend to a considerable extent upon the financial condition of that body. The first activity planned comprises the holding of international meetings at intervals of two to four years, at which papers on subjects in the field of the association's interest will be presented. Hydraulic laboratories will also be invited to present reports on their current research, and this information will be disseminated internationally.

The association will consist of individual members and corporate members. Any person interested in hydraulics may become an individual member: (*a*) if he is professor of hydraulics at a university or engineering college; (*b*) if he is the director of a hydraulic laboratory or holds a responsible position in such a laboratory; or, (*c*) if he holds a responsible position with a firm that manufactures hydraulic equipment or machinery, with a hydro-electric power company, with a governmental organization or with a department of an educational institution having to do with hydraulic design or research. However, the classification given above is not intended to exclude interested persons who may not come under any of the classes listed. The annual dues of an individual member will be 10 reichmarks. Individual members can secure life membership for 100 reichmarks.

Corporate membership is open to: (*a*) universities and engineering colleges that have departments engaged in hydraulic research; (*b*) special institutions for hydraulic research; (*c*) hydraulic laboratories; (*d*) national or other similar committees of the World Power Congress, the International Congress on Large Dams, and the Permanent International Association of Navigation Congresses; and, (*e*) public or governmental organizations that plan, construct or operate hydraulic works. The annual dues of corporate members of the association are set at a minimum of 50 reichmarks.

The officers of the association comprise a president, two vice-presidents and a secretary. Provision is made for a permanent committee composed of the officers of the association and three other members. The permanent committee has the authority to name an executive committee from its own membership.

At the Brussels meeting Prof. Wolmar Fellenius, of Stockholm, was elected president of the association, and a provisional permanent committee was chosen, consisting of J. Blockmans (Antwerp), C. Camichel (Toulouse), H. N. Eaton (Washington), J. B. Egiazaroff (Leningrad), W. Fellenius (Stockholm), A. H. Gibson (Manchester), E. Meyer-Peter (Zürich), Th. Rehbock (Karlsruhe), A. Rohringer (Budapest), F. Schaffernak (Vienna), E. Scimemi (Padua), R. Seifert (Berlin), J. Smetana (Prague), and J. Th. Thijsse (Delft). A provisional executive committee, comprising Messrs. Fellenius, Seifert and Thijsse, was elected.

It is probable that a meeting of the association will be held during 1936 at some central point in Europe, so that the permanent organization of the association may be effected and plans may be formulated for its first activities.

The first step to be taken is to build up the membership of the organization. Since the association will be self-sustaining, it is necessary to depend at least at first, entirely upon dues to finance its expenses. All interested persons and organizations in the United States who wish to join the association are invited to send their application for membership, together with one year's dues (to apply to the calendar year 1936) to the president, Prof. Wolmar Fellenius, Kungl. Tekniska Högskolan, Stockholm, Sweden. No special application form is necessary for this purpose. Further information regarding the association will be furnished in the National Bureau of Standards Hydraulic Laboratory *Bulletin*, Series A, No. IV-1, which will be issued early in January, 1936.

Earthquake-Resistant Construction Applied to California Schools

Seismic requirements of new law effective on 1,001 new school buildings valued at $35,000,000—Fundamental principles in designing to resist horizontal forces—Salutary effect on construction from 2½ years of law enforcement by State Division of Architecture

By C. H. Kromer
Principal Structural Engineer, Division of Architecture, California Department of Public Works

THE LONG BEACH earthquake of March 10, 1933, marked the beginning of a new era in structural design throughout California. Within 30 days after the quake it had been made illegal (by the Field Act) to continue construction practices that had been widely prevalent but were now recognized as extremely dangerous to life and property when earthquakes occur. It is hoped that California's experience, and particularly the recent improvement in design and construction standards, will have beneficial effect elsewhere. What the new laws require, how their provisions have been enforced, and what improvements in design and construction have been developed in these 2½ years of strenuous struggle with the problem of resisting seismic shocks, are outlined in the following. In this outline the author has attempted the presentation of general data developed by the staff that has worked under his direction in the Division of Architecture.

Practically every part of the United States, at some time in its history, has been shaken by a strong earthquake. However, so long as the interval between recurrence of these quakes was as great as a generation no improvement in design or construction practices resulted. Even in localities where shocks occurred more frequently, as in California, progress was extremely slow. In part this resulted from the fact that, although 80 strong shocks have been recorded in California during the past 123 years, within the last score of years only three have occurred in closely built-up areas: San Francisco (1906), Santa Barbara (1925), and Long Beach (1933).

Following the 1906 catastrophe there was much discussion in San Francisco about earthquake-resistant construction. Studies were made, but no permanent regulations resulted. The 1925 Santa Barbara shock convinced engineers, business men and insurance companies that something should be done to protect life and property, and under the sponsorship of the state chamber of commerce joint committees were formed to develop such protection as might be feasible. A most important result of this joint endeavor was the Uniform Building Code, California Edition, produced after years of work by committees representing California sections or chapters of the American Society of Civil Engineers, the American Institute of Architects, the Associated General Contractors, and the Pacific Coast Building Officials' Conference. This long and effective study was a necessary preliminary to the general agreement on the engineering provisions of such a building code that had been reached by the time the 1933 earthquake struck the district southeast of Los Angeles. This time the imperative need for action was all too apparent. Popular opinion at last swung into line with conclusions already developed by thoughtful groups.

School hazards and new laws

The outstanding result of an examination of Long Beach wreckage was recognition of the shockingly dangerous construction used in school buildings—not only in a few cases but quite generally. Fear about schools elsewhere swept through the state like wildfire. Without delay investigations were started earnestly in every city. This spotlight of inquiry brought into sharp relief conditions so dangerous as to beggar description by word picture. There were schools with no anchorage between roofs and walls, or with excessive heavy ornamentation loosely supported over entrances whence it could be tumbled down on outrushing pupils. There was brick construction that had only an outward appearance of strength. All the evidence showed that grave disregard of the fundamentals of earthquake-resistant construction and of requirements for lateral stability was widespread.

Drastic remedial measures were instituted at once. Many schools were condemned or closed temporarily pending repair. In large cities, schools

found safe were used for two sessions daily so that children from other schools could attend. Many classes were held in tents.

The result of lasting significance, however, was the prompt enactment of two measures, the Riley Act and the Field Act. The first of these prescribed minimum horizontal forces (both wind and earthquake) for which virtually all buildings—except farm structures and other minor exceptions—were required to be designed. The second placed upon the Division of Architecture of the State Department of Public Works (referred to hereafter as the Division) responsibility for assuring the safe design and construction of public schools throughout the state. Incidentally, the attention of authorities was directed to their personal responsibility for injury to persons or damage to property in their charge.

Enforcing the new laws

The effect of the Field Act was to load onto the Division, whose engineering staff had shrunk to skeleton proportions during the post-boom days, an enormous amount of work requiring immediate attention. Not only was the Division required to check plans and specifications and assure competent supervision and erection of all new and reconstructed schools, but it also had to examine a very large number of existing school buildings outside the affected area at the request of school trustees who wanted to know the facts about their buildings.

Starting with three structural engineers, the personnel employed in administering the act grew rapidly until at the peak of the initial emergency a force of 75 was necessary to handle the volume of work that poured into the office. In addition, about 468 inspectors have been employed directly by the various school districts to serve under the direction of the architects or structural engineers responsible for the design and under the general supervision of the Division.

Since the new laws were passed and up to Sept. 15, 1935, plans and specifications for 1,001 projects, representing a total value of $35,000,000, had been examined and approved by the Division. Reports also had been submitted on the examination of 956 school plants, having a total valuation of about $75,000,000. The volume of construction in the coming two years should be even greater as more money for essential school housing becomes available; the value of school buildings now projected in the Los Angeles metropolitan area alone is about $22,000,000, and this is in addition to the $12,000,000 that has been spent on schools in that area since 1933. A corresponding volume of school construction work is now being undertaken throughout the state as a whole.

Because the California law is the first, so far as the author knows, to make a serious effort to enforce honesty and competency in building design and construction, an outline of its major features is presented before discussing technical features of its operation. Briefly the requirements of the law are:

1. The Division, under the police power of the state, is empowered and directed to supervise the design and construction of all new school buildings and also to supervise reconstruction, alteration or additions whose estimated cost exceeds $4,000. Before a school district lets any contract for such a project, the written approval of the Division must be obtained. This can be given only after examination of plans, specifications and structural design computations.

2. All plans, specifications and computations submitted with applications must be prepared by a certificated architect or by a person holding a certificate as structural engineer under the California law. No contract for school construction is valid, nor may any public money legally be paid for any labor or material, unless plans and specifications comply with the law and the established rules and regulations.

3. From time to time the architect or structural engineer in charge of the work, as well as the inspector and the contractor, is required to certify that of his own personal knowledge the work performed and material installed conform with the approved plans and specifications. Violation of any provision of the act or any false statement in a certified report constitutes a felony.

4. The Division is required to make such supervision of construction as may be necessary. Adequate, continuous inspection is required during construction by an inspector acceptable to the Division.

State-wide uniformity essential

As a standard of comparison in reviewing designs and examining buildings, the Division has adopted what has become quite generally known as "appendix A" from the standards for structural engineering design and construction set forth in the previously mentioned Uniform Building Code, California Edition. As a basis for adjudging fire and panic safety, the provisions of the Uniform Building Code of the Pacific Coast Building Officials Conference have been adopted. Armed with these two codes, the Division was enabled to take a disinterested stand in the enforcement of reasonable and necessary regulations.

Although authority of the Division in matters of seismic risk is well established, this authority does not extend to matters of fire and panic safety. The Division *recommends* in each case the standard requirements for fire and panic safety. If the recommendations are not adopted, the Division merely goes on record to the effect that its approval of the plans in this case does not extend to fire and panic features.

Fortunately the Division was able to secure the services of a group of competent structural engineers employed under Civil Service to assist in the enforcement of the act. The untiring efforts of this group have been invaluable, as has also been the assistance of the advisory committees nominated by the architectural and structural engineering professional societies and appointed by the director of public works to assist the Division in deciding the many difficult and unprecedented technical questions which arose in the application of the regulations to actual construction problems.

Although earthquake-resistant design had been under discussion for many years prior to 1933, few engineers had had actual design experience along this line because of the reluctance of any community to take the lead in adopting such requirements. (See article by the author on "Design of Earthquake-Resistive Construction," Vol. CXVII, *The Architect and Engineer.*) It was apparent early in the development of the uniform code that such measures could be adopted only as state-wide legislation. Otherwise, any single community setting up special regulations would draw upon itself the implication that it was subjected to an unusual earthquake risk. Fortunately, widespread community interest in seismic risk now is established by high authority. The work of the U. S. Coast and Geodetic Survey, the Carnegie Seismological Laboratory at Pasadena, the Seismological Society of America and other qualified organizations has caused leading seismologists to conclude that there is little if any variation in earthquake risk throughout the state of California. A similar view was presented by Dr. Buwalda, of the California Institute of Technology, chairman of the Division of Geology and Paleontology of the Joint Technical Committee on Earthquake Protection, in a letter to Vierling Kersey, state superintendent of public instruction, in which he said in part:

> The risk at any given point is a summation of the risks involved in possible displacement and earthquake origination from all the spots on all the active faults within destructive radius in the neighborhood. A severe shock on a distant fault may shake down a school just as violently as a lighter shock on a near-by fault, and, since no one can tell where the next displacement on a fault will occur, it is clear that the risk from violent shaking does not differ very much from place to place in the state.

As long as the state maintains basic state-wide requirements, local bodies can and will pass strengthening ordinances; should the state acts in any way be nullified, however, prompt return to the old unsafe methods of construction is likely.

The unnecessary damage and loss of life occasioned by the failure to adopt proper regulations at an earlier date is highly regrettable, but equally regrettable from an enginering point of view is the fact that there were not a large number of properly designed structures side by side with those which collapsed in whole or in part. There were, however, a sufficient number of properly designed and constructed buildings in the affected area of the 1933 earthquake to demonstrate clearly that earthquake-resistant design is practical.

One of the immediate benefits in earthquake-resistant design is the accompanying decrease in insurance rates. The extreme range quoted recently was $3.50 per hundred for the least resistant buildings to 15c. for those properly designed and constructed. An example of this is cited (*ENR*, Nov. 16, 1933, p. 590) by Harold Hammill, structural engineer of San Francisco, who describes the earthquake-resistant design of an eight-story apartment house where the saving in insurance costs more than offset the cost of providing earthquake resistance. This is in line with the reported experience of the Los Angeles School District, which finds that fire-resistant and earthquake-resistant buildings can be built at no increase in cost over less satisfactory types. A less apparent but important saving also results from the smaller upkeep costs of properly designed buildings.

Provisions of the code

Although no requirements not previously recognized as desirable by structural engineers are included in regulations adopted by the Division, this is believed to be the first set of such regulations to be enforced under state law. The provisions, which are covered in detail in "Appendix A," already mentioned, may be summarized as follows:

1. Buildings must be designed to resist horizontal forces proportional to the mass of the building.

2. When structural members are rigidly connected, consideration must be given to the effects of continuity in analysis for vertical as well as for horizontal loading.

3. Consideration must be given to the relative rigidities of adjacent or interconnected parts. An important corollary is that, if the line of action of the resultant of the forces does not pass through the center of resistance, torsional moments will be produced which must be considered in design.

By way of citing typical requirements: every building and portion thereof must be designed and constructed to resist safely within prescribed working-stress values such combinations of dead load, live load and earthquake or wind as will produce maximum stresses. Either wind or earthquake load is to be considered, the design to be for that one which produces the most disadvantageous condition. For buildings not over 60 ft. in height, wind is to be considered as a pressure of 15 lb. per sq.ft. applied in any direction upon the vertical projection of the building normal to the wind direction. For portions of buildings higher than 60 ft. a 20-lb. wind is to be considered.

The minimum earthquake force that must be used is determined by multiplying the actual dead and live-load weight of the building by a seismic coefficient. The Riley Act established this coefficient at a low minimum of 2 per cent; coefficients from 2 to 10 per cent are prescribed for school-house construction, dependent upon variable conditions of soil and location; a value of 10 per cent is rather generally adopted for masonry buildings without frames. There is a strong feeling, however, that these values are low for regions where seismic risk is high. In some locations the minimum should be raised to 10 per cent, with even higher values for specially hazardous parts of the building. A maximum increase of 33⅓ per cent is permitted over the usual design load stresses for combinations of vertical and lateral load.

Some special problems

There are differences of opinion as to the forces which should be considered in a seismic design. Admittedly, slender towers and other unusual structures must be designed on a dynamic instead of a static basis. We cannot predict the nature of the vibrations to which buildings near the epicenter are subjected during an earthquake, but we face the practical problem of designing for reasonable safety.

Since buildings designed on a static basis have survived very severe earthquakes with little or no damage, it is apparent that this system of design is satisfactory for most buildings. However, it does not follow that unthinking application of an arbitrary set of forces will result in a satisfactory design; on the contrary, recognition of their arbitrary nature clearly indicates the necessity for good judgment. For example, a study of such instrumental records as are available, together with observation of the behavior of actual structures, indicates that vertical accelerations near the epicenter are of the same order of magnitude as the horizontal accelerations. Normally, any momentary increase of vertical pressure due to vertical acceleration will be cared for by the reserve strength of a structure, but when the vertical acceleration tends to offset gravity it should not be neglected.

Resonance, fortunately, generally can be considered as only momentary and its effects discounted. When prolonged, it is most destructive.

Many experiments are now being made on the design of water tanks, towers and other unusual structures, and these experiments undoubtedly will lead to fundamental considerations for improved design of ordinary buildings. One of the interesting features brought out clearly by vibrational studies is that transverse, longitudinal and torsional vibrations are independent when structures are truly symmetrical, while in unsymmetrical structures each type of vibration is induced by the other, complicating the situation almost beyond the realm of feasible analysis.

Making provision for lateral forces entails new viewpoints in design. (See article by author: "Structural Problems in Connection With Earthquake Resistive Design," Vol. 24, No. 4, Bulletin Seismological Society of America.) No longer may the interaction of one material with reference to another in the same structure be disregarded even though they are not immediately adjacent. Other assumptions heretofore commonly made are not safe where horizontal forces are involved. Even for purely vertical loads there is usually sufficient restraint to induce flexural stresses and strains in vertical supports; when horizontal forces of considerable intensity are considered, we find that the interaction of the horizontal and vertical members and of such diaphragms as are present, whether trussed or solid, becomes vitally important. We are led to adopt, therefore, a new philosophy of structural design in which analysis is based primarily upon a comparison of deflections and rigidities of the various materials used in any given structure.

This conception has been advanced slowly for some time by engineers in this country and abroad. It is a sound viewpoint and was certain to prevail eventually. However, in advancing this philosophy erroneous ideas had to be overcome, and its practical application was limited up to the time of the Long Beach earthquake. Since then California engineers, especially, have discarded irrational notions about distribution of load and stress and have given consideration to force action as actually distributed. This is particularly necessary in the ordinary buildings with their conglomeration of different materials and corresponding variations in resistance, strength and flexibility. In other words, loads and forces no longer are distributed according to arbitrary rule-of-thumb procedure, but instead a serious effort is made to design for actual load distribution.

Much pioneer work has been done by Japanese scientists in studying the effect of seismic shocks on structures. A discussion of progress and development on this general subject, which duly credited the Japanese, was written by H. D. Dewell of San Francisco and published in *Engineering News-Record*, April 26, 1928, p. 650 and May 3, 1928, p. 699. An outline of "Progress Against Earthquake Hazard in California" was published in *Engineering*

News-Record, July 5, 1934, page 14.

Rigidity is an important item in analyzing the effect of the forces induced by seismic disturbances and their absorption or distribution into the various elements of a building. Consideration must be given to the rigidity of certain unyielding elements and the ability of more flexible elements to deflect enough so that they are freed of load by yielding it to the more rigid elements. This resistance does not depend on initial strength but rather on the relative stiffness or rigidity of the elements or materials present. The stiffer element may not have the requisite strength to resist the induced force safely, but until such time as it does fail it will continue to offer resistance. This holds true even though some much stronger but more flexible element is present and in a position to resist the force if the stiffer element had been absent.

We can go even farther and say that torsional distortion that may possibly result from the premature failure of an unsymmetrically located stiff element may cause failure of a relatively stronger but more flexible frame. This failure might occur even though the frame was strong enough to support the load normally contributory.

School buildings, generally, may be classed as rigid structures in which the attempt is made to connect all parts of the structure in order to preclude, as far as possible, horizontal and vertical differential movements as well as rotational distortion. Three fundamental conditions must be met to attain complete unity, thoroughly knit together all elements of the structure, and at the same time provide structural stability for resistance to lateral forces. These are:

1. There must be vertical elements ample in number and strength to resist forces about both major axes.

2. There must be substantial distributing systems in horizontal planes capable of transferring quickly to the resisting elements the forces set up by earthquakes acting on the building's mass and the mass of the constituent elements.

3. There must be substantial and thorough connection or tying-in of all elements of the structure, including floors, roofs, roof covering, walls, partitions, ornamental features, etc., to the systems which distribute and resist horizontal forces.

Elements of earthquake design

Although the results to be attained are identical, the methods used and the problems involved in obtaining rigidity and structural stability differ widely for the reconstruction of existing buildings and for the construction of entirely new buildings.

New buildings one story in height usually are wood-frame structures. Exterior and corridor walls together with cross-partitions between classrooms supply the vertical resisting elements. Horizontal distribution to these elements is obtained through floors and roof or through trusses provided in these horizontal planes. Of the several methods of making the frames structurally substantial perhaps the commonest is to place the wood sheathing diagonally on one or both sides of studs, nailed at the bottom to wood plates or sills which, in turn, are anchored to the footings or footing walls. At the tops of the studs, that is, at the ceiling or roof lines, continuous plates are provided and proper consideration is given to splicing.

Nailing of sheathing to top and bottom plates is calculated for shear resistance. Sheathing preferably continues in one direction throughout the extent of the wall, and since all sheathing is securely nailed to the studs and blocking as well as to the plates, the part of the sheathing in compression is supported at frequent intervals. Special provisions are required at large openings; sides, tops and bottoms of openings usually have extra studding or blocking, all thoroughly nailed to sheathing. There also must be good anchorage to footing walls. Particular attention should be given to anchorage of sill plates at ends and corners. Overturning must be investigated in the case of short walls comparable to piers.

Another method of bracing such frames is by using diagonal "let-in" wood bracing or blocking (angle bracing). In this case snugly fitting cleats are used at top and bottom of each individual brace. Special attention is given to the nailing to studs and plates so that the vertical-stress components can be transmitted. Diagonal bracing is put in in pairs so that only compressive strength is utilized, and every endeavor is made to develop resistance to vertical and horizontal components of the compression in the angle bracing.

Still another method of stiffening is to use members of metal or wood placed diagonally in pairs as tension members. These may be continuous wood pieces dapped into the studs with substantial connections at the top and bottom. Or, they may be metal rods with turnbuckles, anchored to the footing walls and to vertical and horizontal wall members meeting the diagonals at the top. In the latter case the fitting at the top is made of steel or cast iron; ingenious devices are available to provide for adjustment to varying angles of inclination of the diagonals. Access to turnbuckles is provided so that the rods can be tightened from time to time.

Sometimes turnbuckles are omitted and openings in the footing walls are provided so that nuts over washers can be reached and the rods thereby be kept continuously taut. Continuous steel-strap diagonals have been used in small structures, but these are not satisfactory, since it is difficult to keep them taut enough to act as efficient tension members. Good anchorage to the footing walls is as important in these methods as in the others.

Wood walls and partitions forming the structural resistive members are so attached to the footings and footing walls that the floor framing need be sufficient to distribute only forces of nominal amount to the walls, usually directly to the footing walls. The diagonal subfloor sheathing is attached to blocking so arranged that nailing of blocking to sills is direct and toenailing is avoided. Beams and joists are anchored to walls and tied together at splices. Supporting posts or underpinning are attached to beams by scabs; when tall they are swaybraced.

Horizontal load distribution

The principal horizontal distributing elements occur at the ceiling or roof. Roof sheathing is often made use of by placing it diagonally and providing continuous chords or "moment strips" properly spliced at all resisting walls and partitions. Care is taken to have positive connections to the walls by direct nailing, through blocking if necessary. Solid blocking for rafters is required over supports, as is cross- or X-bridging at frequent intervals between supports.

Other methods of providing roof or ceiling diaphragms consist of horizontal trusses in both directions. Usually they are of limited width but are securely attached to ceiling joists which act as struts or ties connecting the exterior wall to the truss systems. On occasion, diagonally sheathed decks or platforms above the ceiling joists have been used as diaphragms. Vertical bracing systems are provided in the attic spaces to connect all parts of the roof structure to the horizontal diaphragms, whether at roof or ceiling.

For buildings one or two stories in height in which the floor and roof construction is of wood but the walls are of reinforced concrete or brick, it is necessary to provide rigid horizontal distributing systems. A very efficient longitudinal diaphragm system is obtained by using reinforced-concrete corridor floors with reinforcing concentrated at the sides or in the masonry walls supporting it, and connected to transverse walls which extend as partitions between rooms from corridor to exterior walls. It is usually most economical to have these spaced at intervals of 60 to 80 ft., since for corridor slabs of ordinary width (8 to 12 ft.) relatively small deflections of the slabs horizontally can be maintained.

For horizontal forces in the longitudinal direction, lateral beams (wind beams) usually are provided at the exterior end walls and on occasion at interior transverse walls. Above the ceilings of the upper story reinforced-con-

crete slabs are sometimes used, supported by corridor walls. In other cases, truss systems of structural steel supported by the corridor walls are used and the transverse walls are extended to and above the ceilings to furnish reaction supports. Horizontal deflections must be limited, and the truss systems are designed accordingly.

The exterior longitudinal walls are connected to the corridor diaphragms at frequent intervals between the transverse walls by steel or reinforced-concrete members designed as struts and ties. Connections between the diaphragms and exterior walls are sometimes provided by specially designed anchorages through the wood-floor joists, but in these cases the connections are spaced at much shorter intervals. Thicknesses of reinforced-concrete corridor floors acting as diaphragms are governed by shear stresses or by practical limitations, and when thin are reinforced with cross-ribs beneath comparable to stiffeners in steel-plate girders. Care is taken to have the roof structure well tied to the distributing systems of the upper-story ceilings. All ornamental features on facades are securely anchored to the walls.

This type of building with reinforced-concrete walls and corridor slabs is highly efficient because the slabs are inherently capable of horizontal force distribution and afford greater security from fire hazard. For buildings with brick walls similar methods are used; the brick are laid in cement motar, and cement grout is used around steel bar reinforcing. This constitutes the so-called reinforced-brick masonry which now is gradually coming into use.

For new buildings one, two or three stories high and of reinforced concrete throughout the problem is relatively simple since both vertical and horizontal elements can be made to provide for horizontal forces as well as vertical loads. Additional reinforcing is placed in walls around openings. Reinforcement in walls is placed horizontally and vertically, or diagonally in two directions and in two planes. Construction joints are provided with liberal reinforcing dowels and in addition have intermittent keys or interlocking arrangements for shear resistance. Experience has shown that keys of raised type are most efficient and also afford easier cleaning of joints. Generally the keys consist of special concrete blocks embedded for half their depth into the tops of poured walls before the concrete sets.

Rehabilitation problems

In the redesign or rehabilitation of existing buildings to secure structural stability for earthquake resistance, the principles are the same. In the case of wood-frame buildings walls are usually reinforced with diagonal sheathing, additional anchorage of floors to footing walls is installed, and lateral or horizontal systems are provided in attic spaces.

The most frequent problem in rehabilitation concerns the typical two-story building with brick bearing walls and wood floors. Several methods are used to give the exterior and interior walls adequate resistance. For walls 13 in. thick it is customary to peel off or strip one brick width (4 in.) and replace this with reinforced concrete or gunite of the same thickness. In addition, continuous vertical chases 4 in. deep are provided at intervals not to exceed 24 times the membrane thickness. These are reinforced as columns so that in effect there are columns 8 in. deep at intervals of 8 ft. In addition to stiffening the wall membranes, the columns help to combine and support the retained brick. Where it is necessary to remove all vertical loads from existing brick bearing walls, floor and roof joists are supported by beams attached to the columns.

Reinforcing brick walls

The reinforced membranes sometimes used to replace brick on exterior walls occasionally are placed inside, in which case the problem of floor support is simplified. Sometimes these are placed on the outside, and it is necessary to provide brackets to support beams carrying floor and roof joists and rafters. Special attention is given to cleaning brick surfaces against which concrete or gunite is to be placed because surfaces free of mortar or dust are essential to good bond. Sandblasting is the commonest method, although cleaning sometimes is done with electrically operated rotary wire brushes.

When concrete is used in thin membranes, special precautions are taken to insure perfect bond to brick. Concrete is poured in lifts not to exceed 3 ft. with a rather wet consistency (slump of 7 to 8 in.), and care is taken not to spatter brick surfaces during concreting.

For walls 17 in. thick the chases are made 4 in. deeper; in addition, buttons, keys or plugs are provided by removing brick pockets between the vertical chases at vertical intervals of 3 to 4 ft. These pockets are about 12x17 in. in size and are of the same depth as the vertical continuous chases. Chases are made with widths that are one and one-half the depth for gunite work, and reinforcing bars are kept a minimum of 1¼ in. from brick surfaces. Round bars are preferable to square bars for gunite work.

Usually the concrete corridor slabs in such buildings require strengthening, particularly at the longitudinal edges. The customary method is to prepare the under surface of the slab at the edges, install longitudinal reinforcement and encase in gunite. In attic spaces, reinforced-concrete slab or structural-steel diaphragms are added, again providing connections (struts and ties) to exterior walls.

Conclusions

In conclusion it should be emphasized that rational design for horizontal forces does not work an undue hardship on the designer, nor need it increase the cost of properly constructed buildings; it does call for the highest type of professional service. Designers have found that when properly laid out and connected no more material is required than formerly went into less satisfactory and more hazardous types of construction. The most important consideration is the provision of a definite path by which the horizontal forces originating throughout the building can be transferred to the foundations. This is accomplished by the provision of distributing systems located at floor and roof levels that will deliver the forces to adequate vertical elements, keeping in mind that forces are resisted by all elements in proportion to their stiffnesses.

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Roadbuilding Program Shows Heavy Employment

THE ANNUAL REPORT of the Bureau of Public Roads, U. S. Department of Agriculture, for the year ending June 30, 1935, stated that directly and indirectly federal highway activities accounted for more than 5,000,000 man-months of employment. Road work for the year was financed almost entirely with funds from the grant of $400,000,000 allotted for roads by the Recovery Act, which had not been expended in the previous fiscal year and to which was added $200,000,000 under the Hayden-Cartwright Act. The average full-time direct employment on road work involving federal funds was 182,605 men, or more than 2,000,000 man-months, while indirect employment averaged more than 3,000,000 man-months.

The public-works highway program, which was initiated in 1933, had by June, 1935, completed construction of 24,600 miles and had 8,529 miles under construction. Of the mileage completed and under construction, 17,341 miles were on the federal-aid system, 2,413 miles on extensions of the system into and through cities, and 13,376 miles were secondary or feeder roads. The program also included 8,727 miles of substantial work-relief highway construction, which was financed by the PWA and the FERA. Over $33,000,000 was allotted to highway work by the PWA.

Slag-Cement-Blend Concrete Tried at Norris Dam

IN ORDER to determine whether an admixture of slag cement to normal cement in mass concrete structures is advantageous in respect to temperature rise, volume constancy, workability and durability, a full-scale trial was begun by the Tennessee Valley Authority at Norris Dam during the past fall. One of two 50-ft. closing sections near the right bank, from which the stream had been diverted while the main part of the dam was being built, was made of the normal concrete used in the body of the dam, while the other was made with a blended cement consisting of 25 per cent of slag cement and 75 per cent of normal cement. The conditions of the experiment and some of its early results are given below, from a report made by Barton M. Jones, TVA construction engineer on Norris Dam.

The cement normally used in building Norris Dam is Type B modified portland, which is a moderate-heat cement (C_3A content of 8 per cent maximum, C_3S from 35 to 55 per cent, and specific surface by Wagner turbidimeter from 1,600 to 2,200 sq.cm. per gram). The slag cement is an interground blend of granulated blast-furnace slag and hydrated lime. Their principal constituents are shown in Table I.

TABLE I—CONSTITUENTS OF SLAG CEMENT AND STRAIGHT CEMENT

Item	Slag Cement Block 43	Type B Cement Block 43	Type B Cement Block 40
Loss on ignition	2.52	0.85	0.94
Insoluble	...	0.21	0.22
S	1.18	...	...
SO_3	0.24	1.62	1.69
SiO_2	32.11	21.54	21.54
Fe_2O	0.55	4.80	4.84
Al_2O_3	9.83	5.53	5.58
CaO	51.06	63.64	63.66
MgO	1.84	1.86	1.82

Of the two closing blocks it was decided to make Block 40 of normal dam concrete (Type B cement), and Block 43 of the slag-cement blend. In proportioning the concrete mixes, 1 bbl. or 4 sacks of mixed cement, weighing 362 lb., (one 80-lb. sack of slag-cement to three 94-lb. sacks of normal cement) was used in Block 43 to one 4-sack barrel of 376 lb. of normal cement used in Block 40. All other features of the blend concrete were identical with those of Block 40 and other parts of the dam. The two cements for the blend were weighed in separate batchers at the concrete-mixing plant, and were mixed for 2½ min. like all the other concrete.

Test samples of the field mix were made to give data on crushing strengths. Temperatures in the interior of the two dam blocks were measured by electrical thermometers. The behavior of the two concretes in mass form can be compared directly, since the rate of placing concrete in the two blocks was identical and the relation of construction progress to that of adjacent blocks was similar in the two cases.

The two types of concrete were proportioned for the same workability. Their water requirements were practically the same (Table II). It was found that, when the sand gradation was coarser than desirable, the blended cement produced a slightly more workable concrete with less water gain than the straight cement; when the sand gradation was finer than desirable, their difference in workability was small and neither concrete showed appreciable water gain.

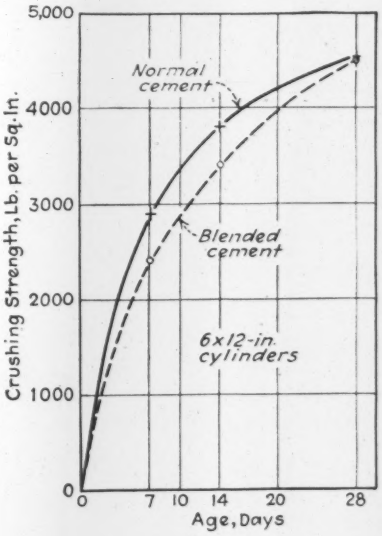

FIG. 1—CRUSHING TESTS of molded concrete cylinders showed that a slag-cement blend gives slower strength gain than a straight Type B cement, but reaches normal at 28 days.

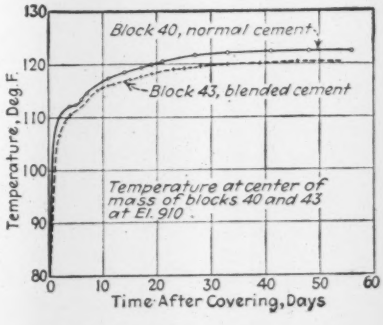

FIG. 2—TEMPERATURES at the centers of the blocks differed by the same amount as the placing temperature, showing that the temperature rise of the blended cement was the same as that of straight cement.

Cylinder strengths up to 28 days (Fig. 1) showed a slower gain of strength by the concrete made of blended cement, but at 28 days it had about reached the normal concrete. Other specimens are to be broken at later ages.

Temperature measurements in the dam blocks (Fig. 2) show about 2 deg. lower temperatures for the concrete made of blended cement, but as the placing temperatures also were lower in this block, the temperature rise was about the same for both cements. Mr. Jones states that on present indications in mass concrete the blend cement produces slightly less temperature rise than the straight cement, but that for accurate comparison it will be necessary to make temperature studies that take into account the effect of surrounding masses of concrete.

Small vertical temperature cracks appeared in the faces of both blocks, but no satisfactory comparison of temperature cracking has been possible up to the present.

TABLE II—MIXTURE DATA FOR MASS CONCRETE
(3-Yd. Batch)

	Straight Type B Cement		Blend of Type B and Slag Cement
Cement content	0.90 bbl./cu.yd.		0.90 bbl./cu.yd.
Cement content	1,015 lb./cu.yd.		977 lb./cu.yd.
W/c by weight	0.67		0.68
	Proportional Parts	Lb.	Lb.
Fine sand	0.25	2,829	2,842
Coarse sand	0.13	1,471	1,478
Fine rock	0.14	1,584	1,592
Medium rock	0.14	1,584	1,592
Coarse rock	0.13	1,471	1,478
Cobbles	0.21	2,376	2,388
Total aggregate	1.00	11,315	11,370
Type B cement	....	1,015	761
Slag cement	....	....	216
Water	....	680	664
Batch weight	....	13,010	13,011

Note: Other concrete mixes used in Block 43, all containing the same blend, are:
Upstream Face Concrete: 1.10 bbl. cement per cu. yd; w/c by weight, 0.59; 6-in. maximum aggregate.
Downstream Face Concrete (in spillway section): 1.20 bbl. cement per cu. yd.; w/c by weight, 0.57; 6-in. maximum aggregate.
Reinforced Concrete (in spillway training wall); 1.33 bbl. cement per cu.yd.; w/c by weight, 0.56; 3-in. maximum aggregate.

Water Stored at Aswan Dam

Land available for agriculture in Egypt will be substantially increased as a result of the second heightening of the Aswan Dam, according to a report by the Department of Commerce. The increased volume of water will irrigate an additional 290,000 acres in the North Delta area. About 20,000 acres will be reclaimed each year, so that the project will take fifteen years to accomplish. Because of an abnormally great flood this year and last year, the Ministry of Public Works proposes to increase the capacity of the Aswan Dam. This dam was built to store 100,000,000 cubic meters of water, and the new works, when completed, will have a capacity of 110,000,000 cubic meters.

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Dec. 19, 1935

The Cost of Ceilings

THOSE who analyzed the breakdown of costs on the Knickerbocker Village housing project, published in the September issue of the *Monthly Labor Review*, must have become bewildered. What was this, an apartment house exclusively for the kiddies? For in plain unmistakable figures the costs were given as 46.8c. per cu.ft. and $2.43 per sq.ft., indicating a ceiling height of not over 5 ft. The reply to our inquiry was equally bewildering; we learned that the "square-foot costs were based on the combined area of floors and ceilings." It is disheartening to realize that for years the building industry has overlooked this simple method of reducing square-foot costs. Why ceilings alone were added to the customary floor surface without including the walls and partitions is not apparent. It is comforting, however, to learn that the Bureau of Labor Statistics has concluded to use hereafter only floor areas in figuring square-foot costs on similar projects. In the Knickerbocker Village costs, as we have abstracted them, the ceilings are omitted.

Mechanical Sewage Filtration

RAPID FILTRATION of settled sewage without the aid of chemicals, adopted for the Goshen, Ind., treatment plant described in this issue, commends itself to attention by virtue of its simplicity and apparent practicability as an economical scheme for increasing the purification capacity of a sewage sedimentation plant. Heretofore practice in connection with rapid filtration included the use of coagulants as a necessary adjunct for successful and efficient operation, but recent semi-plant-scale studies demonstrated that approximately 20 per cent additional suspended solids could be removed from pre-settled sewage merely by passing it through a shallow filter medium. However, the merit and performance of this mechanical aid can be determined only under normal working conditions in a full-sized plant. Operating results from Goshen will be a worthwhile contribution to our knowledge of sewage-purification methods.

Let's Have Details

SINCE the inception of PWA it is claimed by the Division of Investigations of the Department of the Interior that $64,000,000 has been saved through the activities of the division in uncovering fraud, collusion and other irregularities and in finding lack of economic soundness of projects or inadequate financial ability of the proposed borrower of PWA funds. This surprising total is broken down into $29,020,547 for uneconomic projects eliminated and $35,429,380 for savings through detection of fraud and collusion in contract bids and awards and through misrepresentation and other irregularities. The first amount may be classed as an indictment of the PWA's own staff, which is largely an internal affair; but the second sum, until its details are known, stands as a serious indictment of the construction industry. What are the details? The amount saved is given with such exactness as to indicate that it was carefully itemized. Fraud and collusion in handling of contracts is a serious charge; misrepresentation and irregularities are terms that may or may not be serious. Violation of some unimportant PWA regulation might be classed as an irregularity by an over-zealous investigator. If the government stood to lose such large sums through fraud and collusion, why have the conspirators not been prosecuted? The construction industry is entitled to know in detail what items are on the list that make it add up to $35,429,380.

Highway Terminal Problems

IN THE ATTACK on the most difficult element of our road transportation system, the arterial route through town or city, bypasses are finding widespread application in disposing of through traffic. Some other device is essential, however, to take care of the heavy traffic streams that daily enter and leave a large city along its principal radials. It is here that the traffic problem is rapidly becoming more difficult. Express streets, parkways and elevated highways are being developed as solutions, but at very high cost. Such works as the Meadows Viaduct approach to the Holland Tunnel, chief entrance to New York City from the west and south, and the Grand Central and Triborough approaches at the eastern edge of the city, exemplify the size and cost of these traffic entrance facilities. The St. Louis express highway, an equally radical attack on the problem, though adapted to its individual circumstances, involves much lower cost. But it too suggests the large expenditures that will be necessary before we can have adequate terminals. It should be noted especially that all the efforts to provide arterial entrances to large cities remain incomplete, since they fall short of delivering the traffic to its ultimate distribution area. When the planned St. Louis extension eastward to the river is built, or the New York arterial entrances are extended to give effective traffic distribution, a truer picture of the cost of highway terminal service will be had. For the present it can only be estimated that these terminal expenditures will rival those of the open road itself.

Architectural Fire Hazards

LONG AGO some discerning engineer pointed out that fire safety requires more than incombustible construction, and that under the right conditions a fireproof building may be a first-class furnace. Just such conditions are provided by the massing of combustible contents in a location that invites neglect of precautions and lacks adequate protection, and on this score a building designed with dark cubbyholes where combustible material is likely to be accumulated presents a substantial fire hazard. The potential danger became a reality in Washington last week when fire broke out in the new General Postoffice, one of the government's latest buildings, hailed as an architectural achievement because (or in spite of?) its bizarre layout. Visitors to the building will recall the blind sixth story, where for the sake of architectural fancy the outside wall was built without windows. Endless rows of rooms along this wall are dark as dungeons, and quite naturally have been used to store miles of wooden shelving piled high with papers and books; inept arrangement of lighting fixtures makes

necessary to use hanging electric bulbs, and, worse yet, there are no automatic sprinklers. Such spaces are natural fire hearths. The fire may therefore be charged to bad architecture rather more than to chance, for the particular chance that will start a fire may come in many ways and at any time. Unfortunately the postoffice does not stand alone as an architectural hazard. Most or all the government buildings of the new Pennsylvania Avenue group follow the same general design. Had a little less fine-arts thinking been applied to them they would probably be safer; and perhaps they would also be more modern, more practical and more pleasing.

Labor's Responsibility

WHEN WILL CONSTRUCTION succeed in ridding itself of jurisdictional disputes and strikes? This intolerable evil has brought losses of untold extent to worker, to industry and to the public. Beyond question it is the most serious disease that afflicts construction, and until it is eliminated no improvement of labor relations can be hoped for.

Quarrels by various labor unions over their rival claims to control and perform some particular operation have a long history, and in all that history no one instance is known where they have done good to anyone, or served any useful purpose. They originate wholly with labor; neither contractor nor engineer, owner nor inspector, has a part in bringing them into being or can contribute to their settlement. In order to remove them labor itself must take the question in hand and work out the solution.

Jurisdictional disputes exist because the traditional crafts and their craft organizations have not kept pace with the advance in invention and technique. Crafts and unions congregate around characteristic tools—as the trowel, or the saw, hammer and plane—or in some instances according to the materials on which they work, as sheet metal or pipe. But neither tool nor major material is applicable when new forms of construction or assembled units are introduced. When metal window frames and sash replace those of wood, the carpenter's craft skill no longer finds application in setting and fitting them, and several different crafts might claim to be equally equipped.

Of course these controversies develop only in the highly organized fields of construction; general engineering construction, whose working forces are in the main not unionized and not adapted to such organization, knows little of this type of trouble but succeeds in dovetailing operations together and applying new devices and methods with little interference by the internal bickerings of labor groups. The solution of the problem thus is largely at the door of building labor organizations.

The most promising effort to deal with jurisdictional disputes, the well-remembered Board of Jurisdictional Awards, in which various construction interests served with labor, went to pieces after very few years of success because some powerful union groups were dissatisfied with certain decisions. Though all groups involved in any particular controversy are jointly interested in the finished product of the operation, they proved to be so divided organizationally that their private interests dominated those of the product or job as a whole.

It is this misadjustment of organizational form and the personal ambition and striving accompanying it that is mainly responsible for generating and perpetuating jurisdictional group disputes. The political rivalry of labor leaders undoubtedly is to blame for a large part of the property loss, the deaths and injuries and other consequences that have grown out of jurisdictional strikes. Better organization of labor, with more real democracy and less despotism in management, seems essential to elimination of the evil.

Labor relations represent the greatest of the difficulties faced by the construction industry in its recovery from depression conditions. Inequities in wage and working conditions, high cost of construction, unstable employment, and retardation of the natural technologic progress of the art as a result of labor disputes, are separate aspects of the general problem. Improvement of labor relations is in all respects the most urgent need of the industry. But elimination of jurisdictional strike problems must precede all other steps toward reform, and this is the responsibility of labor itself.

Contracting Looks Ahead

READJUSTMENTS and new problems are in sight for the contracting industry as the shift of activity from public works to industrial and residential rehabilitation goes forward. For months past the latter type of work has been growing consistently, and indications are that its rise will continue, while on the other hand public construction faces a decrease with the closing of PWA activities. The two kinds of work normally engage different engineers and contractors, and in part draw on different sources of materials.

This shift presents a new situation for the contractor, and it suggests the wisdom of close study of conditions and careful planning so that he may fit his operations to the change. Farsighted men already have been looking forward to the change. The problem involved is essentially individual, too much so for any general prescription; it must be considered by each contracting unit—and for that matter by every engineer as well—according to his own conditions. Nevertheless, certain general considerations may profitably be kept in mind.

In this shift of construction activity quite obviously efficiency of service and a record of successful and satisfactory work will be controlling factors of success. This is a time, then, when it will pay the contractor to set up his record in ready and complete form, so that it can be quickly and effectively presented and readily verified, for his actual working results will carry more weight than claims and bids. Speed of performance and firm dependability will be quite as weighty factors as a long record of experience in the particular field of operations that a new job may involve, for every potential client knows that modern contracting practice does not confine skill in handling work to one narrow line.

Resourcefulness and adaptability will pay the contractor even larger dividends than usual, for these qualities will be more heavily drawn upon when he takes up a new type of work than when he operates in an accustomed field. At the same time it is essential that he shall have a clear understanding of the difference between public and private contract relations, and of the supreme position of integrity and personal responsibility in determining his success in private construction. In short, the highest virtues of the contracting profession are called upon to assure success in a change like that now in progress.

CURRENT NEWS

Public Works Bill To Be Submitted To Next Congress

Washington Correspondence

THERE will be a public works bill submitted to Congress by the Administration, according to a statement made by President Roosevelt on Dec. 17, in reply to an inquiry concerning funds to continue work on the canal across Florida. The bill will be for something less than $500,000,000, chiefly to cover projects already under construction or authorized. It will call for specific appropriations for specific projects, indicating that the PWA would not be included, as practically all PWA operations are non-federal undertakings for which Congress could not make specific appropriations. However, the President, when asked whether this meant that the PWA was out, said that he did not say that.

As to the Florida canal, the President stated that it was a major public undertaking which should be authorized by Congress and that the public works bill would call for a specific appropriation to continue the work, thus giving Congress an opportunity to pass upon it. Approval or disapproval of the Passamaquoddy power project is to be obtained in the same manner.

Bids Opened in Chicago For Randolph Viaduct Work

Bids on the caisson work on the Randolph St. viaduct over the Illinois Central and Michigan Central railroad tracks to the Chicago outer drive were opened by the Chicago park district on Dec. 10. A low bid of $174,840 was submitted by Mackie, Thompson & Tamm, Inc., Chicago. PWA has made an allotment of $315,000 to cover part of the cost of the Randolph St. viaduct (*ENR*, Dec. 5, p. 795). Work on the outer drive improvements, which had been suspended for some time, was resumed under a recent PWA allotment of $2,324,181.

New Bids Called for Intake For Montreal Water Supply

New bids have been called by the Montreal Water Board for the proposed new intake works in the St. Lawrence River, at Ville Lasalle, on the basis of one combined bid for both the manufacture and installation of the reinforced concrete pipe lines as shown in the original design of the board's engineers. The project is described on p. 849 of this issue. The bids are due Dec. 27 and are to be opened Dec. 30. No further mention has officially been made of the proposal submitted with the original tenders to construct a tunnel intake.

Three Works Projects in Canada Abandoned by New Government

The recently elected King government in Canada has announced the definite abandonment of three large public works projects begun by the government that has gone out of power. The projects are the vehicle tunnel to the airport in Toronto Harbor, $985,000; reconstruction of Alexandria Pier, Montreal, at a cost of $1,392,000; and an archives building costing $358,000 for the federal government at Ottawa. Work on all three projects was stopped as soon as the new government came into power.

The reason given for abandonment of the projects is that they do not provide a large amount of employment, and the government needs the money for other undertakings.

PWA Claims Large Savings Through Fraud Detection

A saving of $35,429,380 since inception of the public works program two years ago by the detection of fraud and collusion in contract bids and awards, misrepresentation and other irregularities is claimed by Louis R. Glavis, director of the Division of Investigations. Surveillance by the Glavis organization, which numbers 320 agents and employees, has also resulted in eliminating projects, valued at $29,020,547 and previously approved on customary examination by PWA, that, under closer examination, betrayed a lack of economic soundness or inadequate financial ability by any of the sponsors applying for PWA funds.

The activities of the Division of Investigations, in preventing substantial losses to the government, covered investigation of collusion and fraudulent bidding involving contractors and subcontractors, irregularities in the employment of labor and the use of insufficient or inferior construction materials, code violation, underpayment of employees and misconduct of PWA officials and employees. The savings effected also include penalties for violation of the eight-hour law on federal projects, reduction in allotments where it was found that the borrower had included excessive amounts in estimates covering overhead and engineering fees, reduction of allotments due to fictitious estimates and excessive appraisals covering the purchase of project sites and savings effected by requiring a change in the method of construction from force account to a contract basis. Reimbursements to labor of $258,331 resulted from investigations of the "kickback racket" where wages were wrongfully withheld by contractors or were repaid by employees.

The administrative expense of the Division of Investigations, from its beginning to June 30, 1935, was reported as $1,337,350, or a fraction over 2 per cent of the amount saved to the government by its activities.

Government Asked To Cooperate in Shore Protection

REPRESENTATIVES of shore interests, coastal states and communities present at the meeting of the American Shore and Beach Preservation Association at Washington on Dec. 16 advocated federal cooperation with local interests in shore protection as the only means of preventing progressive destruction of land by the inroads of the sea. Further, a long-range program designed for integral treatment of extended stretches of shore was declared necessary for effective protection, on the ground that experience has shown any purely local protective work to be destructive to adjoining reaches of shore. Instances of such damage along the northern coast region of New Jersey were cited in support of this view.

Shore line receding

Five technical papers were presented to the meeting. Senator George L. Radcliffe, of Maryland, in his address set forth the long history of destruction of land areas along Chesapeake Bay, which he said has gone on ever since the white man came to the region. He estimated the recession of the shore line at 2 ft. per year. Many islands in the bay on which houses and villages formerly existed have almost disappeared. Protective construction by the local owner is not generally feasible, and a serious question of public policy is involved.

Major Charles H. Cunningham, in an illustrated paper on experimental researches at the hydraulic experiment stations of the War Department, said that as yet no shore protection problems have been investigated, but several studies of inlets and harbors are being planned, including Barnegat Inlet, Aransas Pass and others. He described in some detail the model experiments to develop methods of stabilizing the Southwest Pass of the Mississippi delta for deepwater navigation.

Southwest Pass, the site of Eads' famous jetty enterprise, has since 1902 been found to be "hydraulically perfect," inasmuch as it requires no maintenance expenditure and yet holds its depth of 30 ft. Present commerce demands a deeper channel, however, and accordingly the improvement of Southwest Pass has been begun.

Long-range planning needed

The point of view of the contractor on coastal and waterfront work was presented by Capt. Thomas A. Scott, chairman of the board of Merritt, Chapman & Scott Corp., New York. Captain Scott said that an all-inclusive long-range program of coast preservation is needed, both to develop adequate construction procedure and to assure efficient planning. As to construction procedure, he stated than "on coastal and waterfront work a contractor could effect vast savings if there were a

well defined program of shore and beach protection. Such a program would justify a contractor in securing a source of continuous supply for those materials most needed for shore work.

In a brief address Gen. George B. Pillsbury, acting Chief of Engineers, emphasized the fact that the beach is not fixed but is constantly variable and shifting; it is a river of sand that must be supplied from somewhere. Citizens think of it as fixed and frequently build their houses on the sand. Any structure on the beach, moreover, is likely to have serious effect on adjoining areas by interfering with the movement of the sand. In recognition of this fact, the last River and Harbor Act provided that reports on rivermouth or harbor improvements shall consider also the effect on the shore ten miles to either side.

General Pillsbury expressed a view in opposition to the frequently-heard demand for federal financing of shore protection work, and remarked that the idea has swept over the country that the only agency with visible means of support is the federal government.

A recreational asset

A discussion of beaches as a recreational asset was presented by George Hjelte, superintendent of the playground and recreation department of Los Angeles.

In discussion of the papers, strong pleas were made for immediate federal financial aid in planning and carrying out shore protection work. Statements of recent storm damage along the North Atlantic coast were presented by state and municipal representatives, chiefly of New Jersey. It was pointed out by Col. E. I. Brown, Corps of Engineers, that present law makes the technical staff of the engineer corps of the army available for study and planning of shore-protection work on request of any interested community and at the community's cost, and that the only remaining problem is that of financing the work by the local interests. J. Spencer Smith, president of the association, pleaded that local resources have reached their end and that shore protection must now be taken in hand by the federal government in the interests of preserving the property of all the people. A resolution was adopted by the association calling for an allotment of $25,000,000 of federal emergency funds to shore protection.

Gen. Lansing H. Beach, former Chief of Engineers, emphasized the need for long-continued study of shore-protection problems and methods before any large expenditures for works can safely be undertaken. He further warned against efforts to place the full financial responsibility for shore erosion work on the federal government, and suggested the plan of federal aid to local improvement districts as a desirable plan.

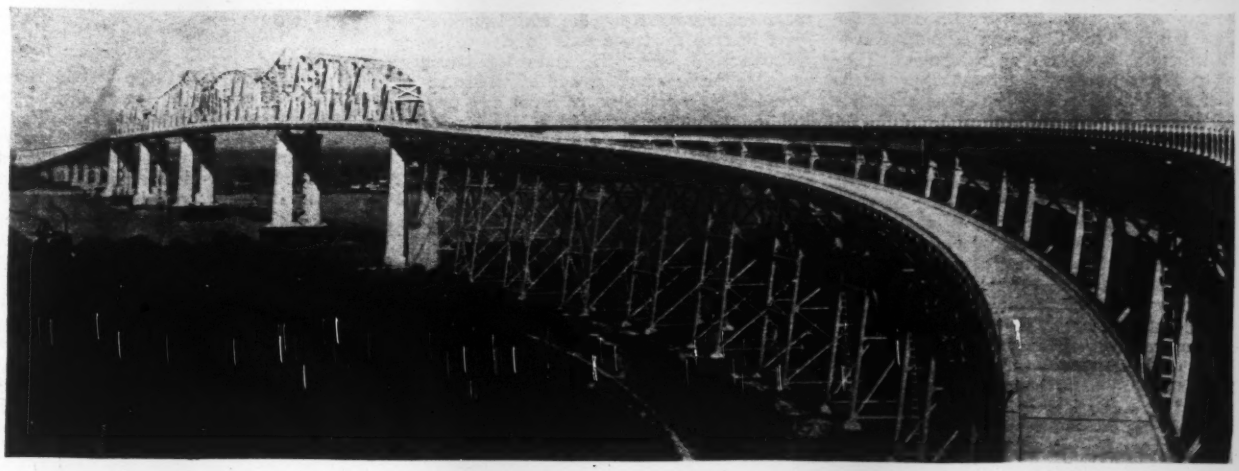
MISSISSIPPI RIVER BRIDGE AT NEW ORLEANS CEREMONIOUSLY OPENED ON MONDAY

The great combined railway and highway crossing of the Mississippi River just west of New Orleans was officially and ceremoniously opened to traffic on Dec. 16 and named the Huey P. Long Bridge. Since Jan. 29, 1933, when construction began, 23,000 ft. or over 4 miles of bridge structure have been built. The main bridge, 3,525 ft. long, includes a 790 ft. main cantilever span, two 530 ft. anchor spans, a 530 ft. through truss span and four deck truss spans; the remainder of the bridge is plate girder approach of unusual length (over 8,000 ft. on either side of the river) made necessary because of the 1.25 per cent railroad grade that must pass over the river with a 135-ft. shipping clearance. The highway approaches are on a 4 per cent grade, so that they are somewhat shorter, begin respectively 3,378 and 3,003 ft.

The bridge carries two railroad tracks within the 33 ft. width between the trusses, and the 18-ft. highway decks are cantilevered on either side. Costing $13,000,000, the bridge is financed by the Reconstruction Finance Corp. whose loan will be repaid half by the state highway department and half by the Public Belt Railroad Commission of the City of New Orleans which will be reimbursed by the Southern Pacific Railroad which will use the bridge on a toll basis. The highway decks are toll free, and the bridge forms an impressive link in the Old Spanish Trail extending from San Diego, Calif., to St. Augustine, Fla.

Most significant from a technical standpoint are the four main river piers which were sunk 170 ft. below water by the sand island method of open dredging using a unique cellular concrete caisson. The main river bridge was erected by guy derricks. Articles covering the design and construction of the bridge have appeared in *Engineering News-Record*, Aug. 17, 1933, p. 189, and May 10, 1934, p. 589.

Modjeski, Masters & Case, consulting engineers, Philadelphia, designed the bridge and supervised its construction with C. Glennon Melville as resident engineer. Moran & Procter, New York, were consulting engineers on the foundations. The substructure for the main river piers was built by Siems-Helmers, Inc., St. Paul. The approach substructure contract was executed by the MacDonald Engineering Co., Chicago. The steelwork of the main river bridge was erected by the American Bridge Co. and of the approaches by the McClintic-Marshall Corp.



Grade Crossing Elimination Ordered in New York State

THE New York State Public Service Commission has directed the elimination of four grade crossings in the village of Lancaster, Erie County, and has directed the completion of the plans for the Boston & Maine R.R. grade crossing elimination in Troy.

Central Ave., which is the principal street in Lancaster, is crossed by the tracks of the New York Central, Lehigh Valley, Delaware, Lackawanna & Western, and Erie railroads. The grade crossings of the New York Central, Lehigh Valley and Lackawanna will be eliminated in accordance with a plan submitted by the village, and the Erie crossing in accordance with a plan submitted by the railroad. The approved plans provide for depressing the grade of Central Ave. on its present location and the plan for eliminating the Erie crossing provides for raising the tracks about 3½ ft. and depressing the street, allowing it to pass below the tracks. At all of the four crossings there will be 60-ft. subways with 40-ft. roadways and two 10-ft. sidewalks. The underpasses will have 14-ft. headroom.

The commission also directed the elimination of the Mechanic St. crossing of the Erie R.R., in Lancaster, by means of a lateral highway north of the railroad running to Cenral Ave. The elimination order also provides for a drainage system for the various underpasses. All of the railroads are to share in the cost of providing the drainage system and the cost of eliminating the New York Central, Lehigh Valley and Lackawanna crossings will be divided among the three companies concerned. Under the law the railroads are required to pay approximately one-half the cost of the elimination and the state and county to pay the remaining cost. The present order merely directs the elimination of the crossings and determines the method to be used and leaves to a subsequent order the allocation of costs and apportionment of responsibility for preparing plans and performing the work.

The commission has also ordered the engineers of the Boston & Maine R.R. to complete in detail their plans for the grade crossing elimination of 24 crossings in Troy. Six different plans for elimination were submitted to the commission, which considers the one submitted by the railroad as the most favorable. Should this plan of the Boston & Maine be finally accepted by the commission, it must also be approved by the city, county and state department of public works.

The plan calls for the elevation of the tracks on a steel viaduct, sufficiently high to clear street traffic without changing the street grade. The estimated cost of this plan is $4,400,410 plus about $9,000 for land damages.

New York City grade crossings

Grade crossing elimination in New York City also has been gotten under way with the beginning of work on the elimination of the grade crossing at 79th St. and Riverside Drive, Dec. 16. This work is part of the general West Side improvement and the northward extension of the present express highway.

The grade crossing elimination provides for a viaduct carrying the street over the New York Central R.R. tracks but underneath the extension of the express highway. The grade crossing structure, after it passes under the express highway, will swing to the south and serve as an approach ramp to the highway. Northbound traffic will enter the highway from a parallel street to be built from 79th to 81st Streets. The cost of the structures is estimated to be about $1,600,000, of which $1,500,000 will be advanced from federal funds that were set aside for grade crossing elimination.

SOCIETY CALENDAR

AMERICAN ENGINEERING COUNCIL, annual meeting and conference of secretaries of Engineering Societies, Washington, D. C., January 9-11, 1936.

ASSOCIATED GENERAL CONTRACTORS OF AMERICA, annual meeting, [illegible] Fla., January 13-15, 1936.

AMERICAN SOCIETY OF CIVIL ENGINEERS, annual meeting, New York, January 15-18, 1936.

AMERICAN ROAD BUILDERS' ASSOCIATION, annual convention and [illegible] show, Cleveland, Ohio, January [illegible]-24, 1936.

CONSTRUCTION EQUIPMENT ASSOCIATION, first annual meeting, Cleveland, Ohio, January 20-24, 1936.

CANADIAN CONSTRUCTION ASSOCIATION, annual meeting, Hamilton, Ont., January 21-23, 1936.

NATIONAL PAVING BRICK ASSOCIATION, annual meeting, Columbus, Ohio, January 29-31, 1936.

ENGINEERING INSTITUTE OF CANADA, annual meeting, Hamilton, Ont., February 6-7, 1936.

AMERICAN SOCIETY FOR TESTING MATERIALS regional meeting, Pittsburgh, Pa., March 4, 1936.

MIDWEST POWER ENGINEERING CONFERENCE, Chicago, Ill., April 20-24, 1936.

NEW YORK SECTION, AMERICAN WATER WORKS ASSOCIATION, midwinter meeting, New York City, December 27.

ASSOCIATION OF HIGHWAY OFFICIALS OF NORTH ATLANTIC STATES, annual meeting, Atlantic City, N. J., February 12-14, 1936.

SOUTHWEST ROAD SHOW AND SCHOOL, Wichita, Kan., February 25-28, 1936.

EXAMINATIONS for registration of professional engineers will be held in the near future as follows:

MICHIGAN—Examinations for registration of Architects, Engineers and Land Surveyors by the Michigan State Board of Examiners for the Registration of Architects, Engineers and Surveyors will be held December 26-28, 1935. Examinations for architects at the University of Michigan; for engineers and surveyors at University of Michigan, Michigan State College and Ironwood.

Control of Federal Buildings May Go to District Inspectors

A fire in the new $10,000,000 Post Office Department building in Washington, D. C., on Dec. 13 resulted in a suggestion by President Roosevelt that fire inspectors of the District of Columbia should have jurisdiction over federal buildings. The district government is agreeable, provided that the federal government bear the cost of increasing the inspection staff.

The fire was confined to one wing on the sixth floor that has no outside exposure due to the deep entablature which belts the exterior of the building at this level. The fire is attributed to contact of a 100-watt lamp with a stack of papers reaching nearly to the ceiling. Because of the fact that the exterior rooms on this floor are without windows, the space is used to a large extent for filing purposes by several government departments.

Extensive damage variously estimated at $60,000 to $300,000 pending thorough appraisal was due largely to the flooding of the building by water. Lack of ventilation made it very difficult to fight the fire.

New Orleans Water Board Terminated by Decree

The services of the present members of the New Orleans, La., Sewerage and Water Board were terminated on Dec. 12 when a judgment rendered by the state supreme court became final and executory.

Earlier this year the state legislature passed a series of laws, one of which removed the sewerage board from city control, and another provided a penalty for the execution of federal projects which were unauthorized by the state. The Sewerage and Water Commission filed action to prevent the carrying out of the state law which did away with the commission as appointed by the mayor of New Orleans and provided for a commission appointed by the New Orleans Port Commissioners, the State Board of Health, the Board of Commissioners of the Orleans Levee District and the Board of Liquidation of the city debt. The state supreme court upheld the state law and the federal Supreme Court denied the application of the mayor and the Board of Liquidation to review the findings of the lower court.

Brief News

AN ALLOTMENT of $10,300 to the Coast and Geodetic Survey of the Department of Commerce for earthquake studies in Montana has been made from old PWA appropriations.

CONTRACTORS bidding for road work in King County, Wash., will in the future be required to include in their figures all necessary highway safety devices, such as warning and stop signs and traffic lights.

WPA DISTRICT DIRECTORS in Oklahoma have received orders to stop the purchase of cement except in emergency cases. These orders came as the federal government launched an investigation of a reported agreement made between cement dealers in the states. It is reported that a survey found the prevailing price per barrel of cement was $2.60 and that cement prices quoted to the government suddenly rose to $3.20.

THE ACT creating the Merrimack Valley Sewerage District which was to collect and treat sewage discharged into the Merrimack River by Lawrence, Lowell and fourteen other Massachusetts cities in the valley, will become void on Jan. 1, 1936, through non-fulfilment of the provisions of the act (*ENR*, Aug. 1, 1935, p. 167).

Detroit Housing Project Fails to Get City Approval

The proposed Chandler Park housing project for Detroit, Mich., received a serious setback on Dec. 10 when the city council refused to vacate six streets running through the site. That the project would, in all probability, be abandoned was indicated by PWA officials. The proposed development would have been made with an expenditure of $4,500,000. Living units for 893 families in three-story apartment and two-story row houses would have been provided by the Chandler Park project. Bids for the construction of the foundation had been opened but were held pending the council's action in vacating the streets.

Projects under contract

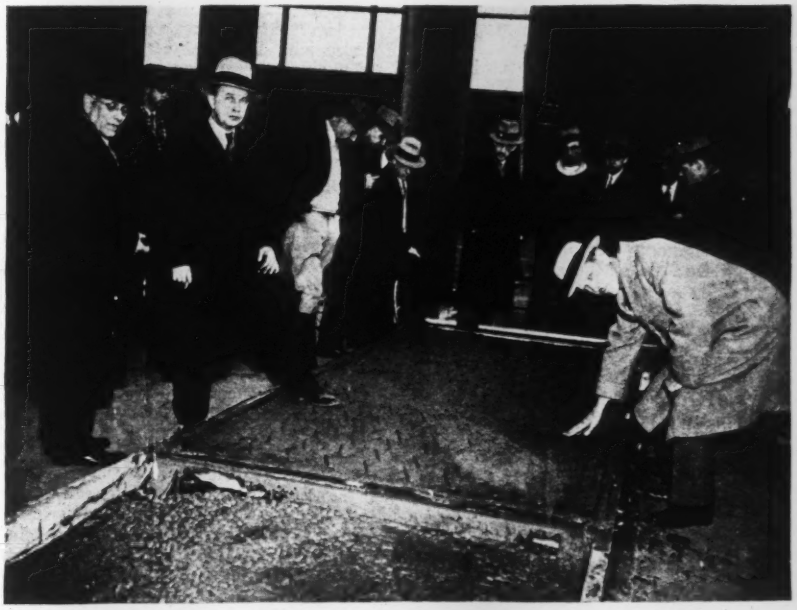
Of the 50 projects in the slum clearance and low-rent housing program of the Housing Division of PWA, 47 were reported as under construction or under contract on Dec. 15. Nine projects are in the course of construction, contracts for general construction have been awarded on three others, foundations are being constructed on 23 projects, and demolition is proceeding on twelve. In addition, seven limited dividend corporation housing projects have been completed under the Housing Division's supervision and are now occupied by about 11,000 persons. A. R. Clas, director, has stated that the entire program will be completed by 1937.

Contract for the general construction of the Stanley S. Holmes Village, Atlantic City, N. J., has been awarded to John McShain, Inc., Philadelphia, Pa., on a bid of $1,195,350. The allotment for this Atlantic City project is $1,700,000. In Miami, Fla., contract for the general construction of the 243 row houses in the 62d St. project has been awarded to Walter Butler, St. Paul, Minn., at a figure of $929,400. The allotment for the project is $1,000,000. The George A. Fuller Co., Chicago, Ill., has been awarded the contract for construction of 518 apartments and row houses in the Park Lawn project, Milwaukee, Wis., on a price of $2,238,500. The allocation for this project is $2,800,000.

Steel Company Plans Extensive Construction

A plan to expend some $40,000,000 for new mills and other improvements has been announced by the Jones & Laughlin Steel Corp., Pittsburgh, Pa. The sum includes $25,000,000 for a strip and sheet mill to be erected in Pittsburgh. The erection and operation of this new mill will mark the entry of the Jones & Laughlin company into the strip and sheet field. The expansion plans call for the creation of a $100,000,000 first mortgage on the properties of the corporation, under which bonds to the amount of $40,000,000 will be issued at present. The remainder of the bonds are to be available for issuance, if required by future needs.

In addition to the $25,000,000 for the new strip and sheet mill, the $40,000,000 bond issue will provide working capital for expenditures on the $5,000,000 electrically driven blooming mill already well under construction and $1,500,000 will be used for additions and improvements for the reducing mill at the Aliquippa works.



SLAB AND COLUMN PRODUCED BY NEW VACUUM CONCRETE PROCESS

A public exhibition of a new process for removing excess water from freshly placed concrete was held at Yale University on Dec. 12, following some two months of investigation and experiment by Prof. Theodore Crane, of Yale, and Duff Abrams, consulting engineer, New York, who have been retained by the inventor, K. P. Billner, of New York.

This special process involves the application of a vacuum to the surface of the concrete through the medium of special pads (in the case of road slabs or bridge decks) or of special forms (in the case of columns). Subjecting slabs of wet concrete (7 gal. of water per sack) to a 20-in. vacuum for 20 min., removes about one-third of the water and renders the concrete stiff enough to walk upon. The 25-in.-diameter and 10-ft.-high column tested was subjected to the same vacuum for one hour, and the forms were removed, with the result shown above. The column was reinforced with a 3-in. spiral and 1½ per cent of vertical steel.

In addition to its primary function of removing the excess water, the process subjects the concrete to compression from the forms, which are under atmospheric pressure on the outside and a vacuum on the inside. The process, it is claimed, will furnish a completed slab surface, eliminating finishing operations and providing any type of non-skid design desired; two types of surface are shown in the above illustration.

The equipment is still undergoing investigation and test, but in principle it consists of a tight outside form, which may be a steel plate or a rubber mat that acts as a seal, and inside layers of wire mesh or porous canvas which will permit the water to be drawn out by the vacuum cups attached over holes in the outside form. A negligible amount of cement is drawn out with the water, although an explanation for this unexpected phenomenon is not yet available. In order to distribute the suction evenly over the surface and not concentrate it at the cups, corrugated metal sheets are used between the inside and outside forms in the case of the metal plates, while for the rubber mats the lower surface of the mats is corrugated.

Cores and specimen cylinders of "vacuum-compression" concrete have shown 7- and 28-day strengths ranging from 53 to 81 per cent greater than for similar concrete not treated. Elimination of shrinkage difficulties is another advantage claimed for the method.

Rail-Water Terminal Bonds Authorized By Bayonne, N. J.

An ordinance authorizing the issuance of bonds for the construction of a rail-water terminal was adopted unanimously by the Bayonne (N. J.) city commissioners on Dec. 14. Under the terms of the ordinance the city will issue about $3,101,000 of bonds to repay the 55 per cent loan portion of a PWA combined grant and loan of over $5,000,000, which has already been allotted for the project. The plan calls for the filling in of about 200 acres of land on the New Jersey side of New York Bay, and the construction of industrial buildings, wharves and warehouses.

Because the immediate issuance of bonds to the authorized amount would increase the debt of the city beyond its legal limit, the commissioners plan to ask PWA permission allowing the issuance of $1,700,000 of bonds at present, pending the retirement of other obligations. As the city then becomes able it will issue more of the bonds. An immediate decision is expected from PWA as the Dec. 15 deadline for having projects under contract is already past. Bids for dredging and bulkheading were called on Dec. 3 and only one was received. The Standard Dredging Co. and J. R. Steers, Inc., both of New York, submitted a joint bid of $1,572,500 for dredging and $1,976,000 for bulkheading. The work includes 8,500,000 cu.yd. of dredging and 10,400 lin.ft. of bulkheading.

The proposed development is to be built on filled land extending from the foot of East 35th St., Bayonne, about 10,000 ft. into the bay. A causeway carrying both rail and highway connections will lead to a filled area 2,100 ft. wide and 4,600 ft. long.

The project is being developed by Central District, Inc., New York City, but in order to obtain funds from PWA the city of Bayonne has agreed to sponsor the project. The arrangements call for city ownership of the land and terminal, which is to be leased to the Central District for a period of 50 years.

Obituary

JOSEPH C. ATWELL died at his home in Detroit, Mich., on Dec. 15. Mr. Atwell, who had represented the A. P. Smith Manufacturing Co., East Orange, N. J., in Indiana and Michigan for the past nine years, was well known in the water works field.

DONALD ALEXANDER MATHESON, British railway engineer, died in Glasgow, Scotland, on Dec. 11, at the age of 75. From 1910 to 1922 Mr. Matheson was general manager and consulting engineer of the Caledonian Ry., and after that and until 1926 was general manager in Scotland for the London, Midland and Scottish Ry.

RICHARD H. TINGLEY, civil engineer and writer, died at his home in Portchester, N. Y., on Dec. 13. He was 79 years old at the time of his death. Mr. Tingley was graduated from Brown University in 1876 and the following year was employed on the construction of the West Shore R.R., and later joined the engineering staff engaged in building the Northern Pacific R.R. in the Yellowstone Valley. He then spent a number of years as a practicing engineer in Providence and New York City.

PHILIP V. R. VAN WYCK, formerly chief engineer of the Metropolitan Conduit Co., died at Winter Park, Fla., on [illegible] 12. Mr. Van Wyck, whose home was in Butler, N. J., was 67 years old at the time of his death. He had been a member of the Corps of Engineers, engaged in surveying a proposed canal through Nicaragua.

CONSTRUCTION STATISTICS FOR THE WEEK

HEAVY PUBLIC AWARDS, topped by the $29,000,000 TVA dam near Guntersville, Ala., push this week's construction volume to $86,817,000, the highest week since March, 1931. This total is made up of private, $5,700,000; public, $81,117,000; federal, $34,580,000 and state and municipal, $46,537,000. This volume is higher than the previous high for state and municipal awards. Corresponding values a year ago are: total, $29,180,000; private, $13,991,000; public, $15,189,000; federal, $4,831,000 and state and municipal, $10,358,000.

The earthwork and waterways classification, at $31,785,000, shares credit for this high volume, with public buildings at $24,442,000. Other classifications are approximately the same as last week except sewerage which is higher, $6,184,000 and bridges, also higher, $4,498,000.

The larger awards for the week include: compressor plant for Industrial Fuel Supply Co., Mountain View, Calif., $300,000; residences, Endicott-Johnson Shoe Co., Johnson City, N. Y., $500,000; school, Hudson, N. Y., $500,000; courthouse, Oklahoma City, Okla., $1,109,000; Archives Bldg., Washington, D. C., $1,323,000; high school, Fitchburg, Mass., $852,000; highways, by Iowa, $584,000; by Michigan, $572,000; by Nebraska, $1,403,000; by Pennsylvania, $600,000; bridges by New York, $1,697,000 and over Missouri River between St. Charles and St. Louis Counties, Missouri highway dept., $1,344,000; intercepting sewer system, Spokane, Wash., $1,750,000; storm and sanitary sewers, relief labor, Stamford, Conn., $1,190,000; dam by Tennessee Valley Authority forces, near Guntersville, Ala., $29,000,000; dredging Calumet and Little Calumet River, $1,894,000; and 70,000 tons of steel rails for Union Pacific R.R., Omaha, Neb., $2,500,000.

State and municipal bond sales mark up the highest volume recorded this year, $39,747,000 which includes a $16,500,000 issue sold by the Port of New York Authority.

CONTRACTS

(Thousands of Dollars)

	Weekly Average Week Dec. 1934	Prev.4 Weeks	Dec.17 1935
Federal Government	$3,979	$3,955	$34,580
State and municipal	15,399	33,078	46,537
Total public	$19,378	$37,033	$81,117
Total private	5,977	7,440	5,700
Week's total	$25,355	$44,473	$86,817
Cumulative to date:			
1934	$1,343,453,000	1935	$1,525,844,000

NEW PRODUCTIVE CAPITAL

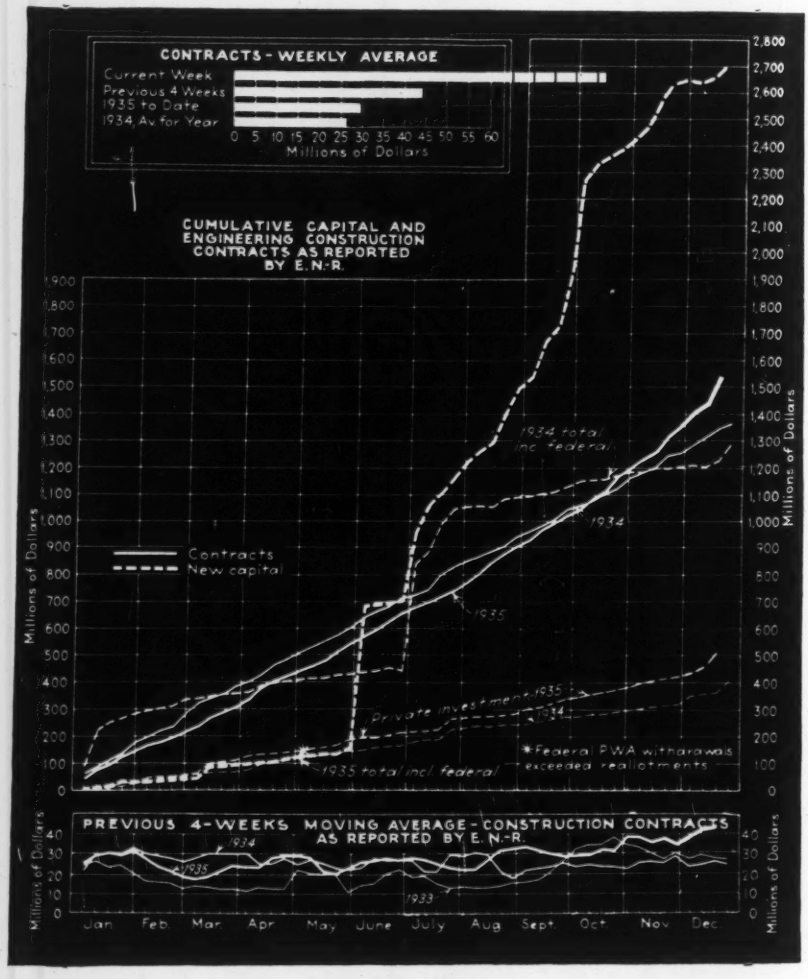
(Thousands of Dollars)

1935	Week Dec.17	Cumulative
State and municipal	$39,747	$471,057
RFC loans, S&M	....	39,380
PWA allotments, S&M	....	761,478
Corporate issues	....	37,018
PWA private	....	9,086
Local contrib. to WPA	....	31,861
Total non-federal	$39,747	$1,349,880
PWA federal	128	1,349,656
Total new capital	$39,875	$2,699,536
Cumulative to date:		
1934	$1,227,367,000	1935	$2,699,536,000

Note: These figures include private bonds, and stocks sold for productive purposes; state and municipal bonds for construction; PWA loans and grants to states and municipalities, including the special highway funds; PWA private loans, and allotments for federal construction, deficiency act funds. An arbitrary percentage (25%) of the WPA allotments and local contributions to WPA work is included to allow for the capital additions through the Works Progress Administration division of the new program.

INDEX NUMBER

ENR Cost	1913 =100	1926 =100	ENR Volume	1913 =100	1926 =100
Dec., 1935	**194.90**	**93.69**	**Nov., 1935**	171	75
Nov., 1935	194.90	93.69	Oct., 1935	162	71
Dec., 1934	201.86	97.00	Nov., 1934	98	43
1934 (Av.)	198.10	95.23	1934 (Av.)	114	50
1933 (Av.)	170.18	81.80	1933 (Av.)	102	47
1932 (Av.)	156.97	75.45	1932 (Av.)	127	56


CONTRACTS - WEEKLY AVERAGE
Current Week
Previous 4 Weeks
1935 to Date
1934, Av. for Year
0 5 10 15 20 25 30 35 40 45 50 55 60
Millions of Dollars
CUMULATIVE CAPITAL AND ENGINEERING CONSTRUCTION CONTRACTS AS REPORTED BY E.N.-R.
Millions of Dollars
1934 total incl federal
1934
1935
Contracts
New capital
Private investment 1935
1934
1935 total incl federal
*Federal PWA withdrawals exceeded reallotments
PREVIOUS 4-WEEKS MOVING AVERAGE - CONSTRUCTION CONTRACTS AS REPORTED BY E.N.-R.
1934
1935
1933
Jan Feb Mar Apr May June July Aug Sept Oct Nov Dec

Current Construction Unit Prices

Montana Highway Construction

THE eight contracts listed below indicate the range of unit costs based on August and October, 1935, awards for highway construction by the State Highway Commission of Montana. The projects used for comparison include twenty treated timber stock passes and bridges shown under Table I, and 30.4 miles of grading, surfacing and bituminous treatment shown under Table II. The total bids of each of the two lowest bidders on the contracts follow:

Stock Passes and Bridges—Contract 1, on Armington-Lewiston highway, three standard treated timber stock passes; two 19-ft. and four 57-ft. treated timber bridges: (A) D. M. Manning, Hysham, Mont. (contract) $14,832; (B) Thos. Stanton, Great Falls, Mont., $14,999. Contract 2, on Miles City-No. Dak. line highway, one treated timber stock pass; one 57-ft. and one 75-ft. treated timber bridges: (C) Prahl & Sawtell, Miles City, Mont. (contract) $6,968; (D) D. M. Manning, $7,320. Contract 3, on Wolf Point-Circle highway, two treated timber passes; one 114-ft., three 76-ft. and one 57-ft. treated timber bridges: (E) H. E. Nesbitt, Miles City, Mont. (contract) $20,450; (F) L. V. Lockwood, Glasgow, Mont., $20,588. Contract 4, on Augusta-Choteau highway, one treated timber pass; one 19-ft., one 57-ft., one 150-ft. and one 200-ft. treated timber bridges: (G) D. M. Manning (contract) $20,201; (H) T. G. Rowland, Salt Lake, Utah, $20,387.

Grading, Surfacing and Bituminous Treatment—Contract 5, Wolf Point-Circle highway, 13.141 miles: (K) Inland Const. Co., Omaha, Nebr. (contract) $142,449; (L) A. Lalonde, Sydney, Mont., $146,645. Contract 6 Augusta-Choteau highway, 10.862 miles: (M) Thos. Staunton, Great Falls, Mont. (contract) $101,576; (N) Clifton & Applegate, Spokane, Wash., $120,364. Contract 7, Madison Canyon-Ennis highway, 2.173 miles: (P) Threet Bros. Const. Co., Lovell, Wyo. (contract) $50,545; (Q) Barnard-Curtiss Co., Minneapolis, Minn., $53,397. Contract 8, Fort Benton-Havre highway, 4.23 miles: (R) E. C. Powell, Missoula Mont. (contract) $31,764; (S) E. N. Brown, Bozeman, Mont., $34,141.

Wards Island Intercepting Sewers, New York

CONSTRUCTION of Section 3 of New York's Wards Island sewage treatment works started in September, 1935, with award of Contracts 3, 4 and 5 for the Manhattan and Bronx grit chambers and appurtenant tunnels, followed in October by Contracts 6, 7 and 8 for the intercepting sewers from E. 73d St. to Manhattan grit chamber, from E. 135th St. to W. 164th St., and from E. 138th St. to Jerome Ave. in the Bronx. The major portion of the work embodied in the latter three contracts comprises 32,000 ft. of sewer, 6,000 ft. of which is open cut; 6,000 ft. rock tunnel, and the balance in soft ground 20 ft. below ground water. The project is financed thru PWA funds, is scheduled for one year's completion and is under the jurisdiction of T. W. Hammond, sanitation commissioner.

The following tabulations list the unit bids and totals of the two low bidders on each of the six separate contracts. Contracts 3, 4, 5 combined: (A&D) Geo. H. Flinn Corp., New York (contract), $2,354,167; Contract 3: (A) Geo. H. Flinn, $1,687,750; (B) Rosoff Tunnel Corp., New York, $1,812,743. Contracts 4 & 5: (C) J. J. Leopold & Co., New York, $672,270; (D) Geo. H. Flinn, $701,418. Contracts 6, 7 & 8 combined: (E) Rosoff Tunnel Corp.

TABLE I—STOCK PASSES AND BRIDGES, MONTANA

Items	Contract No. 1 Quan.	A	B	2 Quan.	C	D	3 Quan.	E	F	4 Quan.	G	H
2. Treated lumber—M.B.M.	104.84	$102.00	$96.00	51.79	$97.50	$105.00	146.87	$100.00	$100.00	158.68	$100.00	$95.00
2. Untreated lumber—M.B.M.	8.53	95.00	96.00	3.71	102.00	95.00	10.56	100.00	100.00	10.44	84.00	75.00
3. Treated timber piles, 20 ft.—each	38	18.00	21.40	.....	.....	.....	47	19.00	20.00	21	17.50	23.25
4. Treated timber piles, 25 ft.—each	112	20.00	24.95	40	21.75	22.00	89	23.00	25.00	54	20.00	26.00
5. Treated timber piles, 30 ft.—each	10	25.00	30.00	20	27.00	26.00	3	30.00	30.00	40	24.00	28.00
6. Treated timber piles, 35 ft.—each	.....	.....	.....	.....	.....	.....	27	33.00	35.00	15	28.00	32.00
7. Structural steel—lbs.	.....	.....	.....	.....	.....	.....	3,310	.15	.10	5,930	.08	.085
8. Gravel sub-base—cu.yd.	.....	.....	.....	40	2.00	2.00	.....	.....	.....	.....	.....	.....
9. Gravel ballast—cu.yd.	77	2.00	2.70	.....	.....	.....	107	2.00	2.00	110	1.40	4.85
10. Remove exist. structures—total	.....	.....	.....	.....	50.00	50.00	.....	75.00	100.00	.....	.....	.....

TABLE II—GRADING, SURFACING, BITUMINOUS TREATMENT, MONTANA HIGHWAYS

Contract No.	5 Quan.	K	L	6 Quan.	M	N	7 Quan.	P	Q	8 Quan.	R	S
1. Excavation, borrow—cu.yd.	224,278	.23	.23	227,059	.19	.225	92,430	.29	.25	38,783	.22	.26
2. Structure excavation—cu.yd.	500	.50	1.00	921	1.00	1.00	212	1.00	1.00	246	1.00	1.00
3. Overhaul—sta.yd.	320,540	.015	.02	174,620	.01	.015	429,180	.01	.01	10,820	.02	.05
4. Riprap, hand laid—cu.yd.	329	3.00	3.00	70	3.00	5.00	205	3.00	2.00	.....	.....	.....
5. Corr. metal pipe, 15-in.—lin.ft.	888	1.15	.75	[1]768	1.70	1.80	144	1.20	1.50	[1]160	1.70	1.75
6. Corr. metal pipe, 18-in.—lin.ft.	264	1.46	1.00	360	2.00	2.30	170	1.90	1.50	.....	.....	.....
7. Corr. metal pipe, 24-in.—lin.ft.	1,930	2.10	1.35	[1]1,340	2.82	3.35	310	2.00	2.25	[1]550	2.82	3.00
8. Corr. metal pipe, 30-in.—lin.ft.	232	2.60	1.70	.....	.....	.....	150	3.15	3.50	.....	.....	.....
9. Corr. metal pipe, 36-in.—lin.ft.	166	3.90	2.70	[1]352	6.00	6.90	50	3.60	4.00	[1]42	6.00	6.50
10. Corr. metal pipe, 42-in.—lin.ft.	178	4.50	3.50	.....	.....	.....	.....	.....	.....	.....	.....	.....
11. Corr. metal pipe, 48-in.—lin.ft.	76	6.10	4.50	[1]36	9.85	11.00	.....	.....	.....	.....	.....	.....
12. Corr. metal pipe, 60-in.—lin.ft.	56	9.25	7.00	.....	.....	.....	66	7.50	7.50	.....	.....	.....
13. Corr. metal pipe, 72-in.—lin.ft.	86	11.70	8.00	.....	.....	.....	.....	.....	.....	.....	.....	.....
14. Gravel sub-base—cu.yd.	30,759	.75	1.10	25,500	.67	.64	5,365	.60	.80	9,661	.80	.70
15. Top course gravel—cu.yd.	11,235	1.58	1.40	10,000	.67	1.00	2,168	1.00	2.00	3,730	1.20	1.10
16. Binder—cu.yd.	2,500	.30	.50	2,100	.20	.30	500	.30	.50	700	.10	.20
17. Binder overhaul—yd.mi.	5,000	.15	.25	4,200	.10	.15	1,000	.20	.25	1,400	.10	.20
18. Water plant—total	.....	300.00	850.00	.....	50.00	200.00	.....	50.00	150.00	.....	50.00	100.00
19. Watering—M. gal.	1,300	4.00	1.50	1,100	1.75	2.00	270	1.50	3.00	425	2.00	3.00
20. Roller—total	.....	300.00	100.00	.....	300.00	300.00	.....	100.00	300.00	.....	300.00	200.00
21. Rolling—hrs.	260	4.00	3.00	225	2.50	3.00	55	3.00	5.00	85	3.00	3.00
22. Prepare roadway—miles	13.141	500.00	50.00	10,703	300.00	350.00	1,794	250.00	400.00	4,246	300.00	300.00
23. Asphalt MC-1 kerosene—gals.	50,793	.115	.12	42,410	.085	.11	9,532	.12	.12	16,441	.09	.11
24. Asphalt road oil SC-4—gals.	50,793	.087	.10	43,471	.075	.09	10,350	.10	.10	16,441	.09	.08
25. Snow fence—lin.-ft.	18,720	.25	.25	1,800	.25	.25	.....	.....	.....	970	.25	.25
26. Wood guide posts—each	259	2.75	1.50	80	1.00	2.00	.....	.....	.....	147	1.00	3.20
27. Round cut slopes—lin.ft.	62,766	.03	.10	60,000	.05	.07	5,495	.03	.10	20,700	.04	.05
28. Gravel stock pile—cu.yd.	1,300	1.58	1.40	1,100	.69	1.00	200	1.00	2.00	425	1.00	1.10

[1]Reinforced concrete.

(contract), $3,967,995, a deduction of $500,000 from separate bids total; (F) Cornell Contracting Corp., $4,594,458, a deduction of $100,000 from separate bids total. Contract 6: (E) Rosoff, $1,461,410; (F) Cornell, $1,622,370. Contract 7: (E) Rosoff, $1,628,090; (F) Cornell, $1,697,568. Contract 8: (E) Rosoff, $1,378,495; (F) Cornell, $1,37[illegible]20.

WARDS ISLAND INTERCEPTING SEWERS, NEW YORK

Item	Contract No. 3 Tunnels Quan.	A	B	Item	Contract No. 4 Manhattan Grit Chamber Substructure Quan.	C	D	Contract No. 5 Bronx Grit Chamber Substructure Quan.	C	D
Test boring—lin. ft.	1,000	$5.00	$8.00	Demolition—total	.....	$5,000	$4,736	.....	$2,000	$800.00
Trench excav.—cu.yd.	30	6.00	6.00	Field office—total	.....	5,000	5,973	.....	5,000	5,925
Excav. above El. 0.0—cu.yd.	2,660	7.45	4.60	Excav. above El. 0.0—cu.yd.	3,300	1.50	4.73	8,000	1.00	2.75
Excav. below El. 0.0—cu.yd.	6,370	7.76	4.60	Excav. below El. 0.0—cu.yd.	13,800	5.00	5.47	8,800	5.00	3.20
Rock boulders—cu.yd.	50	7.00	10.00	Rock excav.—cu.yd.	2,500	4.50	15.55	7,300	4.50	5.48
Shaft rock—cu.yd.	5,475	14.25	38.00	Underpin bldgs.—total	.....	500.00	6,167	.....	5,600	20,800
Tunnel rock—cu.yd.	50,000	12.13	16.00	Steel conc. piles—lin.ft.	1,100	3.00	2.52	1,001	5.00	2.37
Tunnel enlargements—cu.yd.	450	8.75	20.00	Rubbed finish—sq.ft.	31,500	.05	.03	39,000	.05	.03
Steel roof supports—ton	650	85.00	115.00	Reinforcing mesh—lb.	9,000	.10	.06	11,000	.10	.05
Tunnel timber—M.B.M.	10	62.50	120.00	Misc. steel, w.i. iron—lb.	31,500	.15	.08	34,000	.15	.08
Tunnel timber, temp.—M.B.M.	10	52.50	120.00	Floor gratings—sq.ft.	750	2.00	2.18	600	2.00	1.70
Dry packing—cu.yd.	1,500	6.75	5.00	Copper water stops—lb.	4,500	.50	.21	4,700	.50	.18
Pumping—per ft.	30,000	.14	.30	C.I. pipe B&S str.—ton	3	60.00	80.00	4	60.00	79.25
Drainage channels—total	.....	25,000	8,000	C.I. pipe fittings—ton	1	150.00	200.00	1	150.00	138.40
Steel interlining—lb.	220,000	.10	.07	C.I. specials B&S—ton	1	200.00	250.00	1	200.00	173.40
Dry well forms—lin.ft.	300	13.25	9.00	C.I. pipe flged, 12 ft.—ton	5	100.00	175.00	8	100.00	138.50
Chamber forms—lin.ft.	50	30.00	50.00	C.I. pipe flged, to 6 ft.—ton	2	150.00	225.00	3	150.00	195.20
Shaft lining—lin.ft.	1,200	13.00	9.00	C.I. pipe flged, to 3 ft—ton	2	200.00	300.00	2	200.00	239.40
Tunnel lining—lin. ft.	9,000	6.50	3.70	C.I. fittings flged —ton	4	200.00	275.00	4	200.00	223.50
Concrete shaft—cu.yd.	5,000	15.66	10.00	Concrete—cu.yd.	5,000	15.00	11.50	5,500	15.00	12.68
Concrete tunnels—cu. yd.	26,000	13.83	12.00	Thread pipe, 1 in —lin.ft.	100	.50	.50	100	.50	.50
Reinf. concrete above El. 0.0—cu. yd.	200	16.50	21.00	Thread pipe, 1¼ & 1½ in.—lin.ft.	100	.80	.65	100	.80	.65
Drill holes to 1½ in.—lin.ft.	600	1.13	1.00	Thread pipe, 2 in.—lin.ft.	100	1.00	.75	100	1.00	.75
Drill holes to 2½ in.—lin.ft.	1,500	1.35	1.50	Thread pipe, 2½ in.—lin.ft.	100	1.50	1.00	100	1.50	1.00
Grouting pipe—lin.ft.	4,500	.27	.50	Thread pipe, 3 in.—lin.ft.	100	2.00	1.30	100	2.00	1.30
Grouting plant—total	.....	5,000	4,000	Thread pipe, 4 in.—lin.ft.	100	3.00	1.50	100	3.00	1.50
Grout connections—each	1,200	3.38	5.00	Thread pipe fitting, 1 in.—each	5	1.00	.50	5	1.00	.50
Sand, grout—ton	2,000	3.25	3.00	Thread pipe fit. 1¼&1½ in.—each	5	2.00	.75	5	2.00	.75
Grouting—cu.yd.	3,300	9.00	12.00	Thread pipe fitting, 2 in.—each.	5	3.00	1.00	5	3.00	1.00
Portland cement—bbl.	32,500	2.35	3.00	Portland cement—bbl.	7,800	2.50	2.28	8,600	2.50	2.28
Special cement—bbl.	7,000	2.96	4.50	Plug cocks—total	.....	10,000	9,318	.....	10,000	8,890
Alumina cement—bbl.	3,000	7.56	10.00	Sluice gates, etc.—total	.....	22,000	19,847	.....	35,000	29,300
Reinforcing steel—lb.	300,000	.05	.05	Reinforcing steel—lb.	619,000	.05	.038	684,000	.05	.038
Structural steel—lb.	75,000	.06	.08	Structural steel—lb.	58,000	.05	.0398	68,000	.05	.0398
Timber bulkhead, caisson—total	.....	100.00	300.00	Timber bulkhead, etc.—M.B.M.	55	150.00	70.00	37.2	150.00	70.00
Sand fill—cu.yd.	30	1.56	3.50	Grit pumps—total	.....	6,000	7,650	.....	6,000	7,680
Self closing gates—total	.....	14,000	10,000	Grit washers—total	.....	6,000	10,250	.....	4,000	8,230
Timber stop gates—total	.....	5,000	700.00	Timber gates—M.B.M.	4	200.00	140.00	5.3	200.00	140.00
C.I. pipe specials—ton	20	82.50	170.00	C.I. pipe spec. flanged—ton	2	200.00	350.00	2	200.00	281.80
Valves—total	.....	2,000	700.00	Valves—total	.....	1,000	969.00	.....	1,000	805.00
Misc. iron, lead—lb.	25,000	.11	.20	Misc. iron, lead—lb.	22,300	.10	.08	25,000	.10	.08
Gunite coating—sq.ft.	10,000	.25	.40	Grit collectors—total	.....	13,000	15,500	.....	13,000	20,232
Crane—total	.....	5,000	3,500	Chain hoists, etc.—total	.....	1,000	1,571	.....	1,000	1,675
Venturi meters—total	.....	15,000	6,500	Belt conveyor—total	.....	3,000	3,141	.....	3,000	3,100
Sump pumps—total	.....	2,000	3,500	Sump pumps—total	.....	500.00	1,072	.....	500.00	1,252
Clean up—total	.....	19,400	5,000	Bar screens	.....	8,000	14,000	.....	8,000	14,705

Items	Contract 6 Intercepting Sewers —E. 73rd St. to Grit Chamber Quan.	E	F	Contract 7 Intercepting Sewers —Grit Chamber to E. 135th St. Quan.	E	F	Contract 8 Intercepting Sewers — E. 135th St. to W. 164th St. Quan.	E	F
1. Vitrified pipe sewer, open cut, 12 in.—lin.ft.	350	$25.00	$42.00	....	.....	.....	.....	.....	.....
2. Vitrified pipe, sewer, open cut, 30 in.—lin.ft.	80	64.00	47.00	....	.....	.....	910[12]	68.00	42.00
3. Concrete sewer, open cut, 2 ft. 9 in x 2 ft. 10 in.—lin.ft.	1,300	54.00	62.00	....	.....	.....	1,400[13]	51.00	47.00
4. Concrete sewer, open cut, 4 ft. 3 in x 3 ft. 3 in.—lin.ft.	550	82.00	77.00	....	.....	.....	660[14]	79.00	67.00
5. Concrete sewer, tunnel, 4 ft. 3 in x 3 ft. 3 in.—lin.ft.	1,500	75.00	112.00	....	.....	.....	.....	.....	.....
6. Concrete sewer, tunnel, 5 ft. x 4 ft. 2 in.—lin.ft.	750	88.00	117.00	....	.....	.....	.....	.....	.....
7. Concrete sewer, tunnel, 5 ft. 3 in. x 4 ft. 3 in.—lin.ft.	1,660	94.00	127.00	....	.....	.....	.....	.....	.....
8. Reinf. conc. sewer, tunnel, 5 ft. 6 in. x 4 ft. 4 in.—lin.ft.	1,180	127.00	140.00	2,630[1]	184.00	197.00	3,680[7]	160.00	157.00
9. Reinf. conc. sewer, tunnel, 5 ft. 9 in. x 4 ft. 6 in.—lin.ft.	280	129.00	130.00	550[2]	165.00	149.00	1,390[8]	151.00	144.00
10. Reinf. conc. sewer, tunnel, 5 ft. 9 in. x 4 ft. 8 in.—lin.ft.	1,080	131.00	135.00	470[3]	173.00	164.00	520[9]	138.00	204.00
11. Reinf. conc. sewer, tunnel, 6 ft. x 4 ft. 8 in.—lin.ft.	530	129.00	140.00	1,010[4]	167.00	159.00	860[10]	113.00	138.00
12. Reinf. conc. sewer, tunnel, 6 ft. 2 in. x 4 ft. 9 in.—lin.ft.	1,540	129.00	145.00	2,480[5]	141.00	154.00	160[11]	125.00	139.00
13. Reinf. conc. sewer, tunnel, 6 ft. 3 in. x 5 ft. 8 in.—lin.ft.	1,020	125.00	155.00	2,360[6]	134.00	149.00	.....	.....	.....
14. Reinf. conc. sewer, open cut, 7 ft. x 7 ft.—lin.ft.	150	210.00	157.00	....	.....	.....	170[15]	55.00	87.00
15. Fore bay—total	....	63,000	41,332	....	.....	.....	.....	.....	.....
16. Reinf. conc. sewer, open cut, 8 ft. 2 in. x 10 ft.—lin.ft.	100	220.00	200.00	....	.....	.....	.....	.....	.....
17. Manholes, Type A—vert.ft.	100	15.50	34.00	100	20.00	34.00	100	20.00	34.00
18. Manholes, Type B—vert.ft.	100	50.00	38.00	100	20.00	38.00	100	42.00	38.00
19. Manholes, Type C—vert.ft.	300	50.00	45.00	200	60.00	46.00	100	40.00	46.00
20. Manholes, Type D—vert.ft.	100	14.00	28.00	100	15.00	28.00	100	30.00	28.00
21. Manholes, Type E—vert.ft.	200	80.00	34.00	100	20.00	45.00	100	60.00	34.00
22. Manholes, Type F—vert.ft.	300	80.00	44.00	440	80.00	55.00	150	90.00	44.00
23. Earth excav. above elev. 0.0—cu.yd.	200	4.00	4.00	200	4.00	4.00	200	4.00	4.00
24. Earth excav. below elev. 0.0—cu.yd.	200	8.00	8.00	200	8.00	8.00	200	8.00	8.00
25. Rock excav. open cut—cu.yd.	4,600	4.00	3.00	100	10.00	3.00	100	10.00	9.00
26. Concrete, Cl. A, additional—cu.yd.	300	24.00	15.00	150	24.00	15.00	150	24.00	15.00
27. Concrete, Cl. B, additional—cu.yd.	200	20.00	14.00	150	20.00	14.00	150	20.00	14.00
28. Reinforcing steel—M. lb.	800	50.00	60.00	1,107	60.00	60.00	555	60.00	60.00
29. Timber sheeting left in place—M.B.M.	150	60.00	10.00	50	60.00	10.00	700	40.00	10.00
30. Foundation timber—M.B.M.	20	100.00	150.00	10	80.00	150.00	120	80.00	150.00
31. Wood piles—lin.ft.	1,000	1.00	1.20	1,000	1.50	1.20	35,000	.60	1.00
32. Broken stone ballast—cu.yd.	100	8.00	4.00	100	7.00	4.00	500	7.00	4.00
33. Granite block pavement restored—sq.yd.	1,300	5.00	4.00	100	5.00	4.00	100	5.00	4.00
34. Asphalt pavement restored—sq.yd.	100	5.00	3.00	100	5.00	3.00	200	5.00	3.00
35. Concrete sidewalk restored—sq.ft.	100	.40	.30	100	.40	.30	100	.40	.30
36. Bluestone sidewalk restored—sq.ft.	100	.20	.28	100	.20	.28	100	.20	.28
37. Concrete curb restored—lin.ft.	100	3.00	1.50	100	2.50	1.50	100	2.50	1.50
38. Granite curb restored—lin.ft.	100	4.00	2.50	100	3.00	2.50	100	3.00	2.50
39. Bluestone curb restored—lin.ft.	100	2.00	1.50	100	2.00	1.50	100	2.00	1.50
40. Steel sheeting left in place—ton	50	70.00	40.00	....	.....	.....	.....	.....	.....
41. Engineer's field office—total	....	.....	.....	....	.....	.....	.....	15,000	10,717
42. Reinf. conc. sewer, open cut, 3 ft. 9 in x 2 ft. 6 in.—lin.ft.	....	.....	.....	....	.....	.....	260	76.00	87.00
43. Reinf. conc. sewer, open cut, 5 ft. 9 in x 3 ft. 8 in.—lin.ft.	....	.....	.....	....	.....	.....	225	93.00	107.00

[1] 7 ft. 3 in. x 5 ft. 8 in.
[2] 7 ft. 2 in. x 5 ft. 6 in.
[3] 7 ft. x 5 ft. 6 in.
[4] 6 ft. 10 in. x 5 ft. 6 in.
[5] 6 ft. 8 in. x 5 ft. 2 in.
[6] 6 ft. 6 in. x 5 ft.
[7] 6 ft. 3 in. x 4 ft. 3 in.
[8] 6 ft. x 4 ft.
[9] 5 ft. 9 in. x 3 ft. 8 in.
[10] 3 ft. 9 in. x 2 ft. 6 in.
[11] 36 in. pipe
[12] 42 in. pipe
[13] 33 in. pipe, open cut
[14] 36 in. pipe
[15] 3 ft. 9 in. x 2 ft. 6 in